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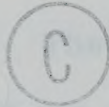
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Measured and predicted performance of deep excavation in
Edmonton Till

by

OLIVER WING HING WONG



A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
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EDMONTON, ALBERTA

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THE UNIVERSITY OF ALBERTA
FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled Measured and predicted performance of deep excavation in Edmonton till submitted by OLIVER WING HING WONG in partial fulfilment of the requirements for the degree of MASTER OF SCIENCE in CIVIL ENGINEERING.

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Date..Nov. 2..1984.....

Abstract

Finite element analysis is used in predicting the magnitude and distribution of lateral pressure acting on deep retaining structures. Not only elastic analysis is performed but also stress path dependent analysis of the soil surrounding the excavation is studied.

The computed results are then compared to the field measurements. Due to the agreement between the computed and measured wall displacements, the computed lateral pressure distribution is inferred to be the actual pressure acting on the wall. Since the calculated lateral pressure showed the semi-empirical design pressure diagram used in practice is conservative, revised design pressure diagrams for the stiff Edmonton till are proposed.

The effect of stiffness of the retaining walls on the wall displacements and lateral pressure is also studied. The tangent pile wall is approximately sixty times stiffer than the soldier pile lagging wall. It can be observed that the pressure resulting from the tangent pile wall is larger than from the soldier pile wall. On the other hand, the calculated horizontal displacements of the walls are not affected by the stiffness of the walls.

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1.0 Introduction

1.1 Nature of the problem

Soil removed from excavation results in stress release within the soil surrounding the excavation. This change of stress causes inward movement of the surrounding ground which redistributes stresses within the soil until equilibrium is re-established. In order to control soil movements around an excavation, supporting structures may be necessary. Before the design of the earth support structure, the engineer must determine the lateral earth pressure that will act on the structural wall, as well as, the magnitude of deformation that will result within the soil and of the wall. If the wall is sufficiently rigid to restrict soil movement, the at rest earth pressure can be used to compute the pressure distribution. On the other hand, if the structure is allowed to move until the shear strength of the soil is fully mobilized, the lateral earth pressure can be obtained from the conventional earth pressure theories. From the point of view of economics and safety, most designs usually fall in between the above two conditions (Medeiros, 1979).

The factors that influence the movements of the soil are the mechanical properties of the soil, ground water conditions, the stiffness of the structural supporting system and the construction procedures. In a braced excavation, due to its supporting condition, the soil

movement is less near the supporting strut. In between the struts, the soil movement may be large (Terzaghi and Peck, 1967). The pressure distribution acting on the structural wall is influenced by the amount of soil movements. The magnitude of the movement also depends on the stiffness of the supporting system. In braced cuts, the supporting system can be divided into the retaining wall and the strutted walls (Medeiros, 1979).

Medeiros (1979) divided retaining structures into three categories according to their rigidity:

1. Rigid wall: the wall is rigid enough to prevent any bending. The movement is either translation or rotation about the toe.
2. Flexible wall: the wall is allowed to bend significantly. The embedment of the wall provides a substantial resistance to the lateral load. Braced sheeting, cantilever and anchored sheet pile walls fall into this category.
3. Semi-rigid wall: the wall is not as flexible as a sheet pile wall but some bending is allowed. Bracing or anchors are present to avoid excessive lateral movement. A diaphragm or tangent pile wall is in this class.

Sometimes, tie backs are designed to replace the struts in a braced excavation. The advantage of using tie backs is to provide more working space within the excavated area. The magnitude of the force within the tie back has an important influence on the soil movement behind the retaining wall.

The difference between using tie backs and struts is that one applies load to the soil and the other resists load due to the soil movements. It is of interest for engineers to investigate how the stiffness of the supporting system influences the soil movements and the pressure distribution that acts against the supporting wall. The supporting system includes the retaining wall, tie back or struts, walers and timber laggings.

1.2 Proposed study

As pointed out previously, the earth pressure is affected by the displacement of the soil. The semi-empirical method of Terzaghi and Peck (1967) for predicting the lateral earth pressure used field measurements to determine the apparent earth pressure diagram. The stiffness of the retaining wall and the supporting system have influence on both the displacements and the apparent earth pressure which Terzaghi and Peck did not account for in their recommendation. Recent research has found that the lateral earth pressure computed by the semi-empirical method when compared to actual field measurements is conservative (Palmer and Kenny, 1972; Morgenstern and Sego, 1981; Sego, 1982; Eisenstein and Medeiros, 1983)

The scope of this research project is to examine the influence of the stiffness of the supporting structure on the measured soil movements and the loads that act on the wall. Only the apparent earth pressure diagram appropriate

for stiff fissure materials is considered in order to limit the study to the soil condition within the City of Edmonton where the field measurements were carried out. A finite element program developed by Medeiros (1979) to predict the soil deformation and stress acting on a braced excavation for the LRT Station in Edmonton was used to perform the analytical study. The research concentrates on the analysis of the soil structure interaction by using the aforementioned computer program and field results that were available.

The analysis undertaken in this report will include both flexible and semi-rigid structures in stiff soils. The results generated by the computer program provide information on the influence that the rigidity of the supporting system has on the lateral wall movements and the apparent earth pressures that would produce these deformations.

As described earlier, the tangent pile wall and soldier pile and lagging wall are considered as semi-rigid and flexible structures. Field measurements were obtained during the construction of the stations of the Light Rail Transit (LRT) system. They were Churchill Station, 104 Street Station and 107 Street Station in which tangent piles were used for the retaining structure. Another field measurement from a soldier pile and lagging wall was obtained at 106 Street and 100 Avenue.

A tangent pile wall is formed by installing circular piles along the excavation boundary. The plan view of the wall can be seen in figure 4.1. For the soldier pile and lagging wall, soldier piles are driven into the soil with equal spacing before the excavation started. When soil is removed, wood lagging were placed between two piles to retain the soil. Tiebacks are always installed when the excavation is deep since a cantilever wall cannot support the earth pressure.

A chapter of literature review will follow this chapter to discuss the earth pressure theories, strutted excavation, the development of semi-empirical earth pressure diagrams in braced cuts, earth pressure on flexible structures and the finite element method. Finally, the computer program developed by Medeiros (1979) is described.

In Chapter 3, the site geology and the local profile are outlined. The construction sequence and the locations of the Light Rail Transit (LRT) Stations and the soldier pile excavation are also contained in this chapter.

The field measurements are outlined in Chapter 4. Measurements of the slope indicators, settlement points, multipoint extensometer and load cell are presented and compared.

The laboratory test results from Wittebolle (1982) and Medeiros (1979) are presented in Chapter 5. These results were used in the finite element analysis of the case studies during this research.

Since the computer program by Medeiros (1979) was originally developed for the analysis of strutted excavation, modifications were made to allow the input of the tie back load for the soldier pile wall. Besides, the wall was installed after the first phase of the excavation. All these modifications are discussed in Chapter 6.

The results of the analytical study for the field case are presented in Chapter 7. The computed deflections, vertical settlements and load cell measurements are compared with the field measurements. The lateral stress distributions from the various assumptions are also presented. Following is a discussion about the comparison of the semi-rigid and flexible wall. Finally, Chapter 8 contains the conclusions of the study and recommendations for modification to the apparent design pressure used in Edmonton along with suggestions for further research work.

2.0 Literature review

2.1 Earth pressure theory

Two classic earth pressure theories for evaluating the lateral earth pressure on retaining structures are in common use. They are the well known Rankine and Coulomb theories.

In 1776 the French scientist Coulomb published a theory of earth pressure that includes the effect of friction between the retaining wall and the soil and which could be applied regardless of the slope of the wall or the backfill. From his observation with dry sand, he discovered that a retaining wall tilts outward until the earth pressure reaches the minimum active state. The theory assumed that the soil slides on the back of the wall and mobilizes the shearing resistance between the wall and the soil. The resultant force is therefore inclined at an angle of δ to the wall, the angle of wall friction with respect to the normal force. The other side of the wedge of material moving downwards behind the wall is a failure surface in the soil which only forms after sufficient shear deformation occurs. Although it can be easily demonstrated that the failure surface must be curved in order to equilibrate the forces, Coulomb approximated the shape of the failure wedge by assuming the sliding surface to be a plane (see figure 2.1).

In 1857 Rankine published a theory of lateral earth pressure pertaining to cohesionless soil with a horizontal ground surface. He realized that a small movement of the

wall will be sufficient to reach that state of stress but he did not mention the magnitude of the movement. Although the equation to determine the lateral load in cohesive material was extended by Resal (1910), it has been called the Rankine theory.

The Coulomb analysis gives an active earth pressure equal to Rankine for a frictionless wall. The assumptions of both Coulomb's and Rankine's theories are that the wall is free to rotate at the toe and no water pressure acts on the wall.

Terzaghi (1941) pointed out that Coulomb's theory is a special case of the general wedge theory. It can be seen that in figure 2.1, the forces meet at one point when the failure surface is curved. From Coulomb's assumption, if the wall is rough, the forces will fail to meet at a single point. Terzaghi was not concerned with the shape of the failure surface or the friction between the wall and the soil. He was interested in the shear strength mobilization along the failure surface. In 1936, he ran large scale model tests using dense and loose sands. He allowed the wall to rotate about the toe and displace in the horizontal direction. The purpose of running these experiments was to investigate the influence of the lateral movement on the shear strength mobilization within the soil mass.

During the experiments on dense sand, a triangular stress distribution was obtained after a large wall displacement. Terzaghi also observed that the initial

displacement changed the initial state of stress from the at rest condition value but without a triangular distribution; the stress near the bottom was decreased and the maximum stress occurred at about $2/3$ from top of the wall. Further movement did not change the total load but caused a stress redistribution within the soil mass. A triangular stress distribution was observed during the entire test for loose sand and larger wall displacement were necessary to reach the values predicted by the Coulomb theory.

2.2 Strutted (braced) excavation

In braced cuts in cohesionless soil using a soldier pile and lagging configuration, piles are driven along the sides of the proposed excavation to the design depth. As excavation proceeds, the sides of the cut between the piles are lined with horizontal lagging boards placed directly against the soil. At a certain depth of excavation, a row of struts are installed. At this stage, the soil movement is so small that no appreciable yielding of the soil mass occurs. Then excavation proceeds to the next set of struts. The rigidity of the struts of the previous level prevents further horizontal movement of the soil at the location of the struts. Under this situation, the lateral pressure caused by removal of soil acting on the lagging and piles will cause a rotation around the previous strut. The second level struts are installed and the process repeated until the final depth of the excavation is reached. Coulomb's and

Rankine's theories cannot be used to compute the lateral earth pressure against a braced cut since the supporting wall was not free to rotate as required by their theories.

The wall deformation is influenced by the stiffness of the wall system, the location of the strut, and the stiffness of the soil mass. Maximum horizontal movements of the wall will occur between the struts and results in a reduction of earth pressure. The mechanism for earth pressure reduction was explained by Terzaghi (1943) as due to arching:

" If one part of the support of a soil yields while the remainder stays in place the soil adjoining the yielding part moves out of its original position between adjacent stationary masses of soil. The relative movement within the soil is opposed by a shearing resistance within the zone of contact between the yielding and the stationary masses. Since the shearing resistance tends to keep the yielding mass in its original position it reduces the pressure on the yielding part of the support and increases the pressure on the adjoining stationary part. This transfer of pressure from a yielding mass of soil onto adjoining stationary parts is commonly called the arching effect and the soil is said to arch over the yielding part of the support. Arching also takes place if one part of a yielding support moves out more than the adjoining parts."

As described by Terzaghi, earth pressure reduces on a yielding portion on a structure but increases on the

adjoining portion.

2.3 Field measurements and the development of semi-empirical rules

2.3.1 Cohesionless soil

Strut load measurements were carried out on an open cut during the construction of the subway in Berlin. The cut was excavated to a depth of about 12 meters in fine, dense, and uniform sand. The pressure distribution converted from field measurements was roughly parabolic. Terzaghi and Peck (1948) proposed the trapezoidal earth pressure distribution of figure 2.2 based on the Berlin data.

Terzaghi and Peck (1967) reviewed their 1947 recommendations. Based on the measurements of strut load of the subway in Berlin, Munich and New York, a pressure diagram was presented by converting the high individual strut loads to an equivalent pressure diagram. The apparent earth pressure was then expressed in terms of the quantity $K_a \gamma H$, where K_a is the active Rankine coefficient $\tan^2(45-\phi/2)$. A uniform pressure equal to $0.65 K_a \gamma H$ throughout the entire depth of a cut in sand was found to embrace the field measurements, as shown in figure 2.3.

2.3.2 Cohesive soil

Measurements in cohesive soils were made during the construction of the Chicago Subway. Terzaghi and Peck (1947)

suggested the stress distribution of figure 2.4. The value of K_a can be computed from Rankine's expression for the active state where the angle of shearing resistance is zero.

Tschebotarioff (1951) questioned the validity of Peck's recommendation that tension will develop to a depth $4C_u/\gamma$. He then proposed the stress distribution of figure 2.5 for clay.

Field measurements for soft to medium clays were obtained in Chicago and Oslo. For the value of N about 4, a plastic zone begins to form in the clay near the lower corners of the excavation. The plastic zone enlarges as N increases and the earth pressure increases. The stability number (N) is equal to $\gamma H/C_u$, and C_u is the undrained shear strength of the soil. Since there is no satisfactory theory developed for calculating the earth pressure against bracing of a cut under this condition, a rough approximation of $K_a = 1 - (m4C_u/\gamma H)$ was suggested. The reduction factor, m , depends on the stress-strain characteristics of the clay soil. Values of m were found to vary between 0.4 and 1.0 for Oslo and Chicago clays respectively. The difference between these two clays was the extent of consolidation. The Oslo clays appear to be normally consolidated whereas the soft Chicago clays was slightly overconsolidated. The strength was not significantly altered by this consolidation, but it increases the initial tangent modulus to a value greater than that of the normally consolidated Oslo clay. The apparent pressure diagram suggested by Terzaghi and Peck

(1967) is shown in figure 2.3. The value of m to be taken as 1.0 except for those normally consolidated clays with N greater than 4 for which m should be less than unity.

It can be noted that when N is less than 4, the assumption of the development of plastic zone is not valid. Measurements were made in cuts in stiff fissured clays in London (Golder, 1948). The value of $K_a = (1 - 4C_u/\gamma H)$ was negative. Terzaghi and Peck suggested the apparent pressure diagram as shown in figure 2.3 with the magnitude range from $0.2\gamma H$ to $0.4\gamma H$ for this type of soil. For temporary support, the lower value may be used. Otherwise, the higher value is recommended.

Terzaghi and Peck's semi-empirical method for both cohesive and cohesionless materials are only applicable to deep braced excavation for which the depth of cut is greater than 6 meters. Also the method assumes the water table is below the base of the excavation.

Strut load measurements were made on the construction of the subway station in Oakland, California. The pressure distribution on the wall was computed by converting the strut loads to equivalent pressures. From the measurements recorded, only the pressure of the strut at the fifth level (a total of 6 levels) was larger than the recommended design pressure. From the observed data, Armento (1972) proposed an empirical lateral earth pressure diagram as shown in figure 2.6 for both cohesive and cohesionless soils.

2.4 Earth pressure on flexible structures

2.4.1 Cohesionless soil

Tschebotarioff and Brown (1948) and Tschebarioff and Welch (1948) published papers reporting the results of a large scale tests with anchored bulkhead sheet pile walls. A triangular stress distribution which agreed with classical earth pressure theories was observed and the bending moment could be predicted from the observed pressure assuming a hinge existed at the dredge level of the bulkhead walls. The authors also observed that no reduction in earth pressure resulted due to the deflection of the sheet pile between the anchor and the dredge level.

Model experiments were carried out by Rowe (1952). In each test, the tie rods were allowed to yield after dredging until maximum bending moment of the wall was reached. A triangular stress distribution was measured. Rowe believed that arching in the soil behind the sheet pile wall was destroyed due to yielding of the tie rod. A decrease in the maximum bending moment with increasing flexibility of the sheet pile were observed during the tests with various wall stiffness. From the fact that the deflection was large near the dredge level and small movements occur at the base of the wall, the earth pressure in these two locations were higher than predicted by the classical earth pressure theories. Since the moments produced by these two increased pressures were in opposite sense of the maximum moment on

the wall, a reduction of maximum moment would result. For more flexible walls, larger deflection were allowed and thus the maximum bending moment was reduced. Brinch Hansen (1953) obtained the earth pressure distribution of a sheet pile wall at failure based on plasticity theory. If this earth pressure was used in design of a sheet pile wall, the tie rod dimensions and the wall section would be approximately the same as if designed using the method proposed by Rowe (1952). The difference between the two methods was Brinch Hansen recommended a smaller factor of safety with respect to penetration depth of the sheet pile wall than Rowe (1952).

In a paper published by Terzaghi (1953), a design procedure for a sheet pile wall was developed based on Rowe's findings. Terzaghi recommended that Rowe's reduction factor for the maximum bending moment could be applied when the sheet piles were driven into a fairly homogeneous clean sand. In loose silty sand, silt and clay, no moment reduction was allowed according to Terzaghi.

Rowe (1956) extended his design method to sheet pile wall at failure by using model experiments. Together with the results reported by Rowe (1957) it was shown that for elastic tie back yields during dredging additional moment reduction occurred due to inward wall movements and subsequent high pressures above the tie back location. This effect, he termed "flexure above the tie level". Thus the effect of pressure redistribution below the dredge level and

the pressure redistribution above the anchor were included in his design procedure. An additional 20% of moment reduction due to arching effect was observed in experiments, but he felt arching was unstable for most type of walls, and therefore should be neglected in design (Rowe, 1957).

2.4.2 Cohesive soil

Golder (1948) reported the earth pressure against flexible vertical walls retaining a stiff fissured clay. Measurements of the pressure in the timbering of a deep excavation behind a retaining wall in the London clay were made. The wall was about 13 meters deep and horizontally supported by struts. The pressure distribution was converted from the loads measured within each struts. According to classical earth pressure theories or Peck's recommendations, the wall would have been able to stand vertically by itself and there should be no load transferred to the wall. The measurements showed the struts carried substantial load.

Another earth pressure measurements made in a braced experimental cut in the stiff fissured crust of the Norwegian marine clay was carried out by DiBiagio and Bjerrum (1957). The opening was about 14 meters long, 0.9 meter wide and 4 meters deep. Six horizontal struts were provided for supporting the wall and load gauges were installed between the strut and the wall. Horizontal and vertical soil deformations were recorded. The loads recorded were then compared to Terzaghi and Peck (1947)

recommendation. It was found that only the load at the upper and lowermost strut were underestimated and the prediction by the aforementioned method was satisfactory.

Tieback loads were measured during the construction of an underpass in the City of Edmonton (Morgenstern and Sego, 1981). The depth of excavation was about 7 meters in stiff fissured clay. Peck's empirical pressure diagram was used to design the supporting system of the excavation. Two rows of anchors were installed to support the sheet pile wall. The load cell measurements showed a decrease in load with time and change in climatic condition; the upper anchors and the lower anchors recorded an average decrease to 33% and 58% of the design loads after about 3 months.

Sego (1982) reported the tie back measurements of an excavation in stiff clay in Edmonton. The design was based on Peck pressure diagram of $0.4\gamma H$ at the mid-height of the excavation where the measurements showed a magnitude of $0.19\gamma H$ from back calculated pressure distribution which is illustrated in figure 2.7.

2.5 Finite element method

2.5.1 Introduction

The finite element method was used to evaluate the stresses and deformations in the soil due to boundary and body forces acting. The problems were solved by the theories of elasticity and plasticity where deformations were small

in the first case and larger in the second. As soils have nonlinear stress strain characteristics, the stress path or plane strain assumption was introduced to the finite element method. This simulated the actual stress strain soil behaviour instead of assuming the soil behaves as a linear elastic material.

In fact, the actual earth pressure on a retaining wall was extremely difficult to determine since it depends on the relative movement of both the soil and the wall. Friction between the wall and soil will be created by this relative movement. The earth pressure will change depending on the magnitude and direction of this mobilized frictional resistance between the soil and the wall.

Finite element analysis using the various assumptions discussed above have been discussed by many researchers in the literature. The results computed have been compared to experimental and field measurements. These will be discussed in more detail in the following sections.

2.5.2 Finite element analysis and experimental results

Finn (1963) discussed the effect of various boundary conditions on the horizontal stresses acting against retaining walls. Finite element analysis performed by translating the wall in the active sense and rotating the wall about the top were evaluated. In both cases, no wall friction and no slip between the wall and soil were considered. The pressure distributions predicted from the

elastic analysis agreed with the empirical distribution suggested by Terzaghi and Peck (1948).

An experiment of a long vertical wall with soil mass on one side was carried out by Girijavallabhan and Reese (1968). A rigid steel plate was used to represent the wall and the soil used was dry Colorado River sand. The results were compared to a finite element analysis using the plane strain configuration. Load verses horizontal movement curves showed good agreement between the dense sand test results and the predicted results from finite element analysis.

The results of an excavation must be independent of the number of steps in the excavation process if an elastic soil was assumed. Christian and Wong (1973) reported that in finite element analysis, many computed results indicated that the displacement at the top of a vertical cut appears to depend on the number of steps used in excavating the elastic soil and thus a problem exists with the use of this method.

A series of laboratory tests on a flexible model tie back wall were carried out to investigate the actual three dimensional nature of a tie back wall (Tsui and Clough, 1974). Finite element analysis were performed under two assumptions:

- a. plane strain loading conditions to represent the actual three dimensional loading by the braces or tie backs; and
- b. continuous planar walls to simulate the behaviour of

discontinuous walls like a H-beam and lagging wall. The results of the finite element analyses accurately predict the deflection of the soldier pile but not the lagging behaviour when compared to the actual test data. This was because the stiffness of the lagging was much lower than the stiffness of the soldier pile and the whole supporting system was represented by an equivalent planar wall in the finite element analysis.

Ground movements of an instrumented excavation in clay was analysed using finite element method under linear, bilinear and nonlinear conditions (Pontes, Filho and Medeiros, (1982)). The vertical displacements predicted from all three assumptions overestimate the field measurements. The linear and bilinear cases over estimated the lateral movements while the nonlinear analysis showed a better comparison to the field measurements. It was also observed from the horizontal displacement curves that a significant difference in the magnitude of displacements below the bottom of the excavation occurred. The reduced as the excavation proceeded.

2.5.3 Finite element analysis and field measurements

Morgenstern and Eisenstein (1970) published a paper discussing the load and deformation due to excavation. Since the deformations were generally small and the soil was stressed in its linear or pseudo-linear range, linear elastic method of analysis were considered as appropriate.

On the other hand, problems involving large deformation where soil yielding was being simulated, non linear analysis was more suitable. Finite element analyses were performed under the assumption of linear elasticity with the following boundary conditions

- a. no yielding of the wall,
- b. the wall yields $0.0025H$ in the active sense, and
- c. the wall yields $0.0025H$ in the passive sense

where H was the height of the wall. The distribution of maximum shear stress for several cases with different boundary conditions were compared. The shear stress distribution for zero lateral yield showed the maximum shear stresses developed when the rigid base was below the level of the excavation were higher than, when the rigid base was located at the excavation base. When the wall was allowed to translate in the active sense, slight changes in shear stress occur when the rigid base was deep compared to when the rigid base was near the base of the excavation.

Foundation settlement or heave predicted by Eisenstein and Morrison (1972) using the finite element method agreed remarkably well with field observations. The modulus of deformation were measured using the *in situ* pressure probe for the analysis of two multi-storey building in Edmonton, C.N. and A.G.T. towers. It was found that the magnitude of moduli obtained with the pressuremeter was significant higher than the value obtained from laboratory tests. Different values of poisson ratio were used in the finite

element analysis. The predicted settlements were found to be affected significantly by poisson's ratio. Maximum settlements of 15 mm, 22 mm and 27 mm with poisson ratio of 0.495, 0.4 and 0.3 respectively, were obtained from the finite element analysis with the same elastic modulus for the C.N. tower. The best fit poisson ratio, μ , in the finite element analysis was found to be 0.42.

The influence of wall stiffness and anchor prestressing on earth pressure distribution in sand was studied by Egger (1972). Finite element analysis on flexible sheet pile wall and stiff cast inplace diaphragm wall were compared based on the soil behaving as an elastic material. The sheet pile wall was 100 times less stiff than the diaphragm wall. Both excavations proceeded in three levels of cuts. After the first excavation stage, the horizontal displacement of the flexible wall was about 3 times larger than the diaphragm wall at the top. A coefficient of active earth pressure of 0.27 was estimated at the upper part of both walls while it approached the coefficient of earth pressure at rest near the bottom of the excavation.

The loads applied to the walls were changed after the final stage of excavation, the horizontal movements at the top of the flexible wall decreased from 25 mm to 17 mm as the anchor load increased from 10 kN to 60 kN. For the stiff wall, it reduced from 23 mm to 14 mm with the same anchor load change. The stiff wall was 100 times stiffer than the flexible wall and the change in horizontal displacements

were only 2 mm and 3 mm with the anchor loads increased from 10 to 60 kN. Therefore the influence of wall stiffness on the displacements was negligible when compared to the anchor reaction.

A soldier pile and prestressed tie back system was employed to support the excavation of an office tower and a low-rise building in clay. Finite element analyses were used to predict the deflections and stresses on the wall and the soil at each stage of excavation (Clough, Weber and Lamont, 1972). The initial at rest earth pressure coefficient was 1.3 and the earth pressure distribution computed was compared to Terzaghi and Peck's (1967) semi-empirical diagram. Good agreement was found before prestressing the last anchor; after prestressing, the pressure at the bottom of the excavation became very high. On the other hand, the wall deformations predicted by finite element analysis overestimated the field measurements. An increase of wall stiffness in the finite element analysis by ten times the original stiffness resulted in a reduction of the maximum deflection from 50 to 35 mm and the maximum settlements behind the wall from 20 to 12 mm. This result showed that the stiffness of the wall affect the horizontal displacement significantly; the settlement reduction was due to the smaller horizontal displacement. As the magnitude of the horizontal displacement was large softening of the soil might occur and a stiffer wall could prevent excess soil movement. When compared to Egger (1972), the horizontal

displacement resulting from his analysis was so small that the soil was within the elastic range. The stiffness of the wall did not have a significant influence on the horizontal displacement such as, occurs when the soil displacement were large as in the case studied by Clough, Weber and Lamont (1972).

An analytical study of a braced excavation in weak clay with a sheet pile wall was conducted using the finite element method to evaluate the influence of variables such as soil strength, deformation characteristics, stiffness of the wall and of the bracing system (Palmer and Kenny, 1972). The influence of the variables were compared by varying the value of one variable while all others were held constant. The results showed that the soil deformation modulus had the greatest influence of the variables. Also pile stiffness and strut stiffness were important factors of the supporting system. A small anchor prestress reduced deflections, but further prestress would not provide additional benefits. The findings in this study showed that the Terzaghi and Peck (1967) design envelope was safe but excessively conservative under some field conditions.

Clough and Tsui (1974) studied the difference between anchored and braced excavation. Smaller deformation of the tie back walls than the braced walls with shallow depth of excavation was found in the finite element analysis. On the other hand, when the excavation was deep, the deformation of the tie back walls became larger than that of the braced

walls.

It was found that the deformation of a retaining wall was also affected by the soil beneath the base of the excavation. Stroh and Breth (1976) showed the results from finite element analysis with non-linear elastic assumption that the wall deformation was twice as much with clay as with rock beneath the base of the retaining wall.

Non linear analysis was performed to compare the measured deformation of the ground and strains that result within sheet piles and struts (Izumi, Kamemura and Sato, 1976). The pressure distribution computed after the final stage of excavation was smaller than the measured value from the top of the sheet pile to the bottom of the trench, while it was almost the same beneath the excavation. The axial load calculated was larger than the measured value in the struts. Agreement on the horizontal displacement occurred between the calculated and the measured values.

Non linear analysis was also used to study the influence of the degree of wall embedment and bracing stiffness upon diaphragm wall behaviour (Martin and Hoatley, 1979). The strut load and the maximum horizontal deflection from the analysis appeared to have no significant dependence on the degree of wall embedment. The analysis also showed the diaphragm wall behaviour was largely influence by the stiffness of the strut and not the wall embedment.

2.5.4 Summary

In order to simulate the actual field conditions in the finite element analysis, many assumptions were made. These assumptions were justified since the finite element results compared well with the field or the experimental measurements.

The factors affecting the results of the finite element solutions include the excavation procedure, boundary conditions, and stiffness of the supporting system. Various assumptions of soil conditions of linear elastic and nonlinear elastic (plane strain and stress path) also significantly affect the calculated result.

In this project, finite element results were used to back analysis the field case records that will be described in the following Chapters. The computer program used throughout this project will be based on the program developed by Medeiros (1979).

2.6 Description of the computer program from Medeiros

2.6.1 Introduction

A finite element program was developed by Medeiros (1979) to simulate various phases of the construction procedure of the Churchill Station. Detail description of this station can be found in later sections or Medeiros (1979). The finite element method has been used extensively by many researchers in analysing different retaining walls

and excavations as previously outlined. The analytical results compared well with the field measurements. The advantage of using the finite element method was its ability to simulate complicate boundary problems, construction sequences and to deal with different constitutive models representing both the soil and the retaining wall.

2.6.2 Stress changes with depth

The initial vertical stress assigned to each element increases linearly with depth below the surface; the initial horizontal stress can be calculated by:

$$\sigma_3 = K_0 \sigma_1 \quad (2.1)$$

where

σ_1 = vertical effective stress

σ_3 = horizontal effective stress

K_0 = coefficient of earth pressure at rest

The loads obtained were removed by dividing the force into a number of steps. In each step, a portion of the load was applied and a new set of stress and strain within each soil element was obtained. From the new stress condition, new elastic constants were calculated and the process repeated until the final step was complete. The program then goes to the next excavation phase.

During the excavation, struts can be installed by changing the stiffness of the elements at their locations to

represent structural material. Besides, if a tie back was to be installed, a force would be applied at the specified node to represent the strut load. The logic of the program is illustrated in figure 2.8.

The program calculates the initial stresses in each element before the excavation begins. The excavation was represented by assigning a small value of the elastic modulus to the appropriate elements and a new modulus of deformation was assigned to the structural elements. The program then calculates the stresses and strains in the soil and the wall. The whole process was repeated until the final phase of excavation.

2.6.3 Load Determination

Ishihara (1970) showed that for soil removed in one cut or removed in several smaller increment, the result were the same for linear elastic materials. It simply means that the incremental excavation procedure does not affect the calculated results inn an elastic material. The introduction of struts or tie backs alter the boundary conditions such that the results from a strutted or tie back excavation will not be a unique solution for different increments since a prestressed load will be introduced to the soil by the tie back whereas the strut will resist the loads resulting from the soil movements. For a realistic analysis, stress path dependency of soil was used in each phase of the excavation. A remarkable difference in the calculated strain was

reported by Clough and Woodward (1967) in an non linear analysis comparing construction of an embankment in one phase and multiple phases. This suggest the need to analyses the excavation using the appropriate number of excavation phases.

The analysis of loads was based on the 'gravity turn on' method. Before any excavation or removal of elements, gravity forces within the finite element mesh were calculated. The excavation was then represented by applying the gravity force with the same magnitude but in the opposite direction. A change of stress and strain was then calculated. This method was simple and straight forward to use. On the other hand, it has some deficiencies since it has a built in coefficient of earth pressure at rest ($K_o = \mu / (1-\mu)$, μ is the poisson's ratio) which cannot be changed.

During the analysis, only one set of elastic constants could be used and this prevents the application of a non linear stress strain relationship. The introduction of incremental analysis solves this problem. The initial state of stress can accommodate any coefficient of earth pressure at rest. Removed elements representing excavation are assigned a small value of elastic constants. From the elements adjacent to the boundary, the nodal loads are determined using those element stresses.

In figure 2.9, a qualitative horizontal stress distribution showed that higher stress gradients were

observed for points closer to the wall (Christian and Wong, 1973) although much reduced by the presence of the wall. It was suggested that when developing a mesh, small elements should be provided close to vertical walls to reduce the distance between the centres of the adjacent elements and the boundary nodes. The determination of the horizontal nodal loads in the program used in this research was carried out by averaging the stresses of the elements to be excavated adjacent to the wall. An increase of 10% of the lateral loads was explained by Medeiros (1979). A test of the analysis procedure for a wall without embedment was carried out. The results showed a difference of 10% between the resultant force calculated from the final stress and the strut load which should equal. Due to this discrepancy, the load should be adjusted. In determining the load at the bottom of the excavation, pairs of elements above and below were averaged to get the vertical nodal loads which agree well with the test results.

2.6.4 Stress strain relationship

The nonlinear material behaviour observed in the laboratory caused the incremental finite procedure to be introduced by Clough and Duncan (1969) using the 'pass stress solution'. The method allows a new set of elastic constants to be determined using the stresses calculated from the previous increment. Therefore, in each increment, the elastic constant could be reduced using this method.

Another method suggested by Kulhawy, Duncan and Seed (1969) and Gopalakrishnayya (1973) reduced the number of iteration but with the same precision and the elastic constants were evaluated by means of average stress strain parameters. The advantage that the stiffness matrix did not have to be inverted for each increment as was proposed by Zienkiewicz, Valliappan and King (1969). This method also reduced the number of iterations and was called the initial stress method or the modified Newton-Raphson method.

The average stress solution required fewer than half of the increments of the past stress solution to get the same solution. The latter was used in developing the program for this research. The advantage of having a constant stiffness matrix as in initial stress method therefore could not be exercised.

The hyperbolic equation (2.2) proposed by Kondner (1963) showed that the non-linear stress strain curves of both sand and clay were best approximated using:

$$\sigma_1 - \sigma_3 = \epsilon / (a + b \epsilon) \quad (2.2)$$

where

σ_1, σ_3 = the major and minor principal stresses

ϵ = the axial strain

a = the reciprocal of the initial tangent modulus

b = the reciprocal of the asymptotic value of stress difference which the stress strain curves approaches

infinite strain $(\sigma_1 - \sigma_3)_{ult}$ as shown in figure 2.10

The coefficient a and b can be determined by transforming the axis by plotting the stress strain data with the following equation:

$$\epsilon / (\sigma_1 - \sigma_3) = a + b\epsilon \quad (2.3)$$

As shown in figure 2.10, a and b were the intercept and the slope of the resulting straight line respectively.

Due to the fact that the hyperbola found is always below the asymptote for all finite values of strains as shown in figure 6.10, the asymptotic value is larger than the failure value. Duncan and Chang (1970) proposed a correction factor R_f to relate the failure stress and the asymptote that

$$R_f = (\sigma_1 - \sigma_3)_f / (\sigma_1 - \sigma_3)_{ult} \quad (2.4)$$

where

$(\sigma_1 - \sigma_3)_f$ = deviator stress at failure

$(\sigma_1 - \sigma_3)_{ult}$ = deviator stress of the asymptote

The value of R_f has been found to be between 0.75 and 1.0 and essentially independent of the confining pressure.

Since divergence will occur as the deviator stress approaches failure, a small value of elastic constant should be assigned. The elastic constants for different stress path are determined by:

1. Passive compression test

Janbu (1963) from his experimental work suggested an equation to relate the initial tangent modulus and confining pressure:

$$E_i = K \sigma_0 \left(\sigma_3 / \sigma_0 \right)^n \quad (2.5)$$

where

E_i = initial tangent modulus

K = modulus number

σ_0 = atmospheric pressure

σ_3 = minor principal stress

n = the exponent determining the rate of variation of E_i with σ_3

The relationship between the coefficient b and confining stress can be expressed in terms of Mohr-Coulomb failure criteria as:

$$b = (1 - \sin \phi) / 2 \sigma_3 \sin \phi \quad (2.6)$$

By combining equations 2.3 to 2.6, a stress strain relationship can be found from the experimental parameters K , n , ϕ and R_f .

Figure 2.11 shows how the modulus of deformation was interpolated linearly between two curves in which the confining pressure falls between the laboratory test curves.

2. Active extension test

Experimental results showed that the origins of the curves were displaced by a certain value of the deviator stress axis due to anisotropic consolidation. Equation 2.2 is then rewritten to relocate the origin.

$$(\sigma_1 - \sigma_3) = \epsilon / (a + b \epsilon) - (\sigma_{1i} - \sigma_{3i}) \quad (2.7)$$

or

$$(\sigma_1 - \sigma_3) = \epsilon / (a + b \epsilon) - \sigma_{1i}(1 - K_0)$$

where

σ_{1i}, σ_{3i} = initial major and minor principal stresses from experiments

The modulus of deformation is determined by following the same procedures described from (1) but using (2.7) instead of (2.2) to relate deviator stress and axial strain.

3. Active compression test

During this test, the major principal stress was kept constant while decreasing the minor principal stress. The fact that Equation 2.5 was no longer valid under this circumstance, but due to the range of the performed tests, the few points falling beyond the limits were close enough to allow the use of the coefficient a obtained from the nearest stress strain curve. The Mohr-Coulomb failure criteria should be expressed as:

$$b = (1 + \sin \phi) / 2 \sigma_1 \sin \phi \quad (2.8)$$

4. Proportional loading active test

From the observation of laboratory tests, the modulus of deformation for deviator stresses below the initial value was close to a constant but followed a hyperbola shape curve after. Therefore, a constant value determined from the experiment was assigned for deviator stress below the initial and thereafter the procedure similar to that described for active compression tests to establish a hyperbola was followed.

Different values of poisson's ratio in finite element analysis result in significant difference in the calculated stress and strain. The value of 0.42 was suggest by Eisenstein and Morrison (1972) for Edmonton till based on back analysis of foundation settlements in downtown Edmonton.

From Hooke's Law, the modulus of deformation for different stress path can be obtained by rearranging the following:

$$\Delta \epsilon_1 = 1 / E \{ \Delta \sigma_1 - \mu (\Delta \sigma_2 + \Delta \sigma_3) \} \quad (2.9)$$

$$\Delta \epsilon_2 = 1 / E \{ \Delta \sigma_2 - \mu (\Delta \sigma_3 + \Delta \sigma_1) \} \quad (2.10)$$

$$\Delta \epsilon_3 = 1 / E \{ \Delta \sigma_3 - \mu (\Delta \sigma_1 + \Delta \sigma_2) \} \quad (2.11)$$

The above equations can be simplified according to the stress path condition. For triaxial passive compression and triaxial active extension:

$$E = \Delta \sigma_3 / \Delta \epsilon_3 \quad (2.12)$$

For triaxial active compression

$$E = - 2\mu \Delta \sigma_1 / \Delta \epsilon_3 \quad (2.13)$$

For plane strain passive compression

$$E = (1 - 2\mu^2) \Delta \sigma_3 / \Delta \epsilon_1 \quad (2.14)$$

For plane strain active compression

$$E = - \mu (1 + \mu) \Delta \sigma_1 / \Delta \epsilon_3 \quad (2.15)$$

Finally, the stress strain relationship between the soil structure interface is still to be determined. Goodman, Taylor and Brekke (1968) developed a finite element program for analysing the behaviour of jointed rock by using one dimensional slip elements. The element accounts for the relative movement between the structure and the ground.

During developing this program, it was observed that no relative movement between the soil and the structure occurred. Therefore such elements to account for the potential relative movement were not necessary.

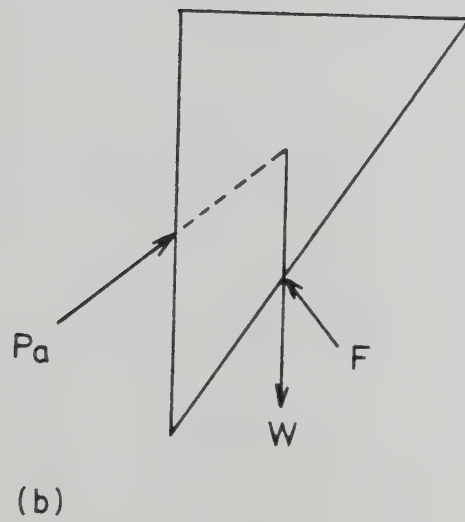
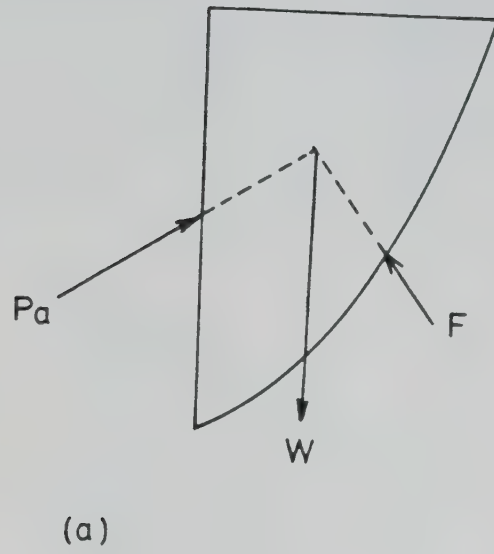
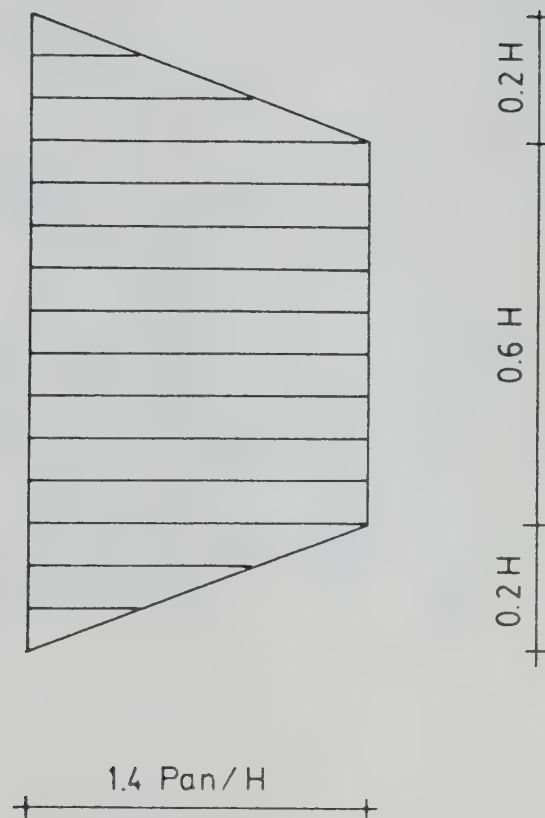


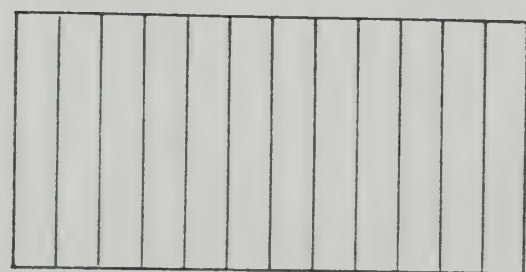
Figure 2.1 Wedge Equilibrium



$$P_{an} = \frac{1}{2} \gamma H^2 K_a$$

SAND

Figure 2.2 Terzaghi's stress distribution in braced cuts

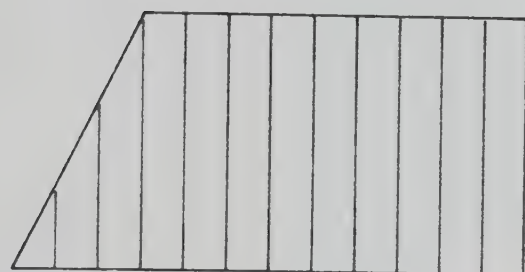


$$0.65 K_a \gamma H$$

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi}$$

a. SAND

H

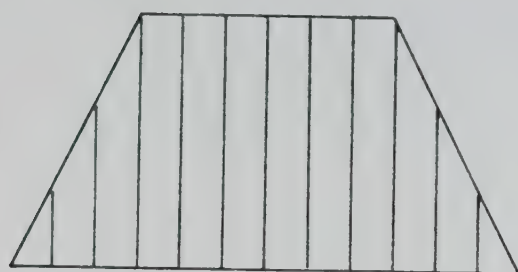


$$10 K_a \gamma H$$

$$K_a = 1 - m \frac{4 C_u}{\gamma H}$$

b. SOFT TO
MEDIUM CLAY

$.75 H$



$$0.2 - 0.4 \gamma H$$

c. STIFF SOIL

$.25 H$

Figure 2.3 Lateral pressure from Terzaghi and Peck, 1967

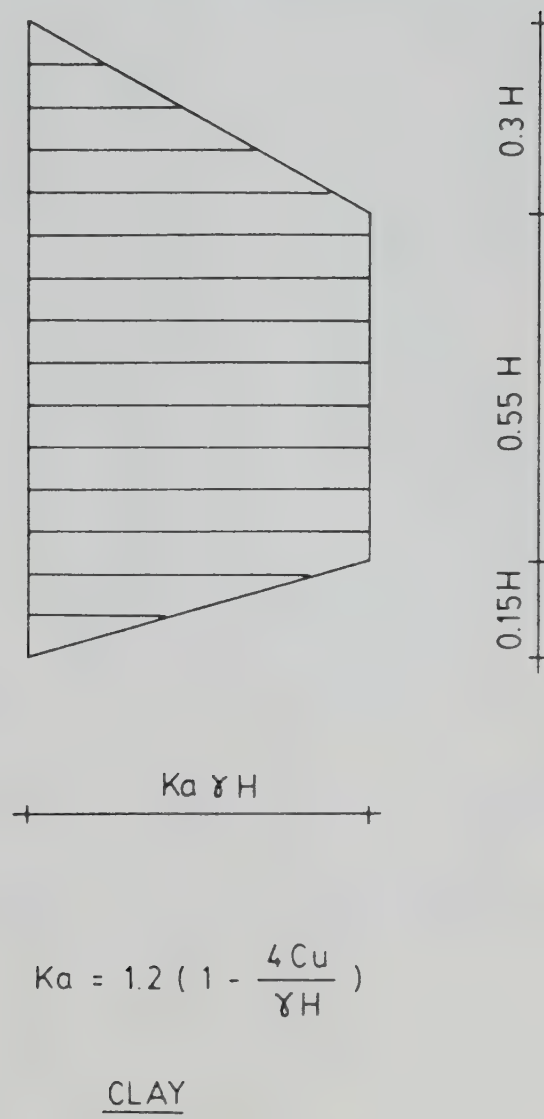


Figure 2.4 Peck's stress distribution in braced cuts

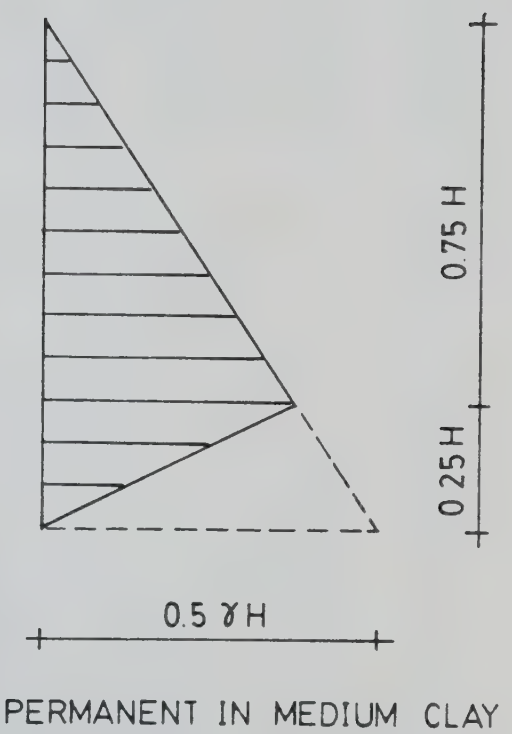
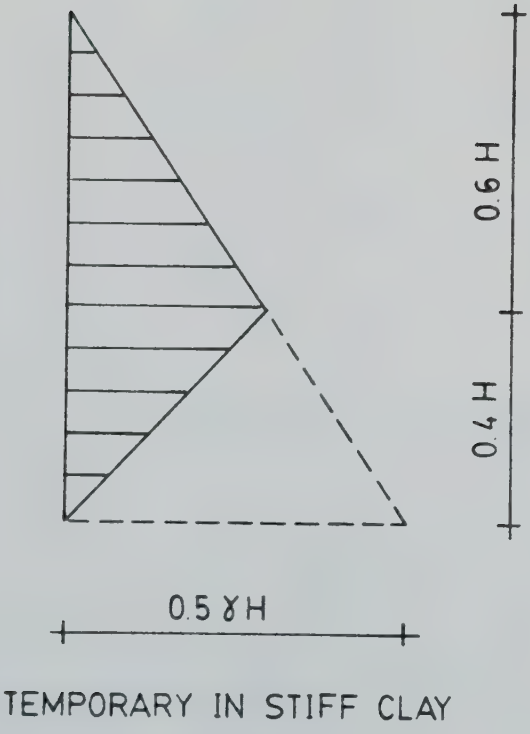


Figure 2.5 Tschebotarioff's stress distribution in braced cuts

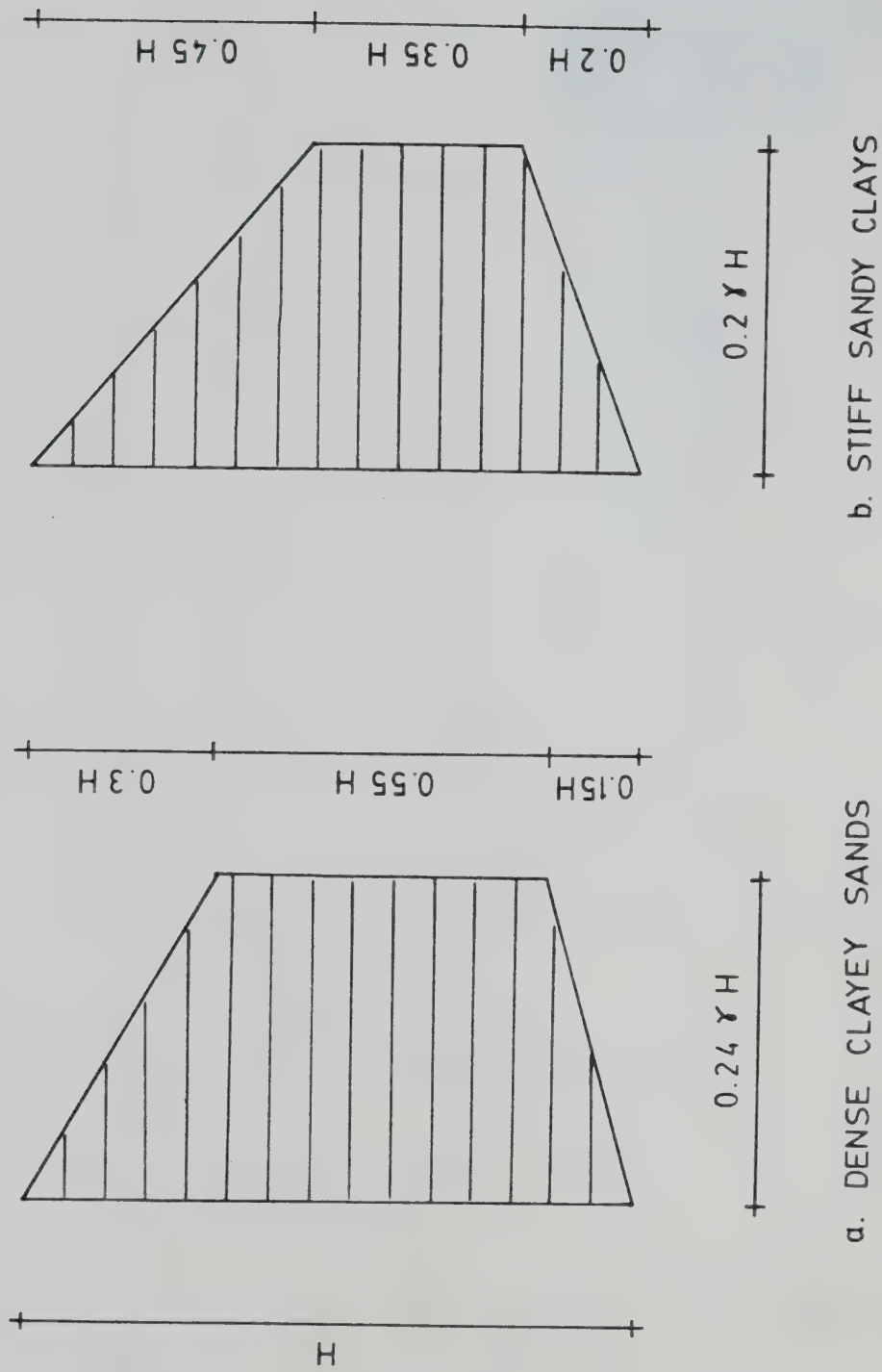


Figure 2.6 Lateral earth pressure diagram by Armento, 1972

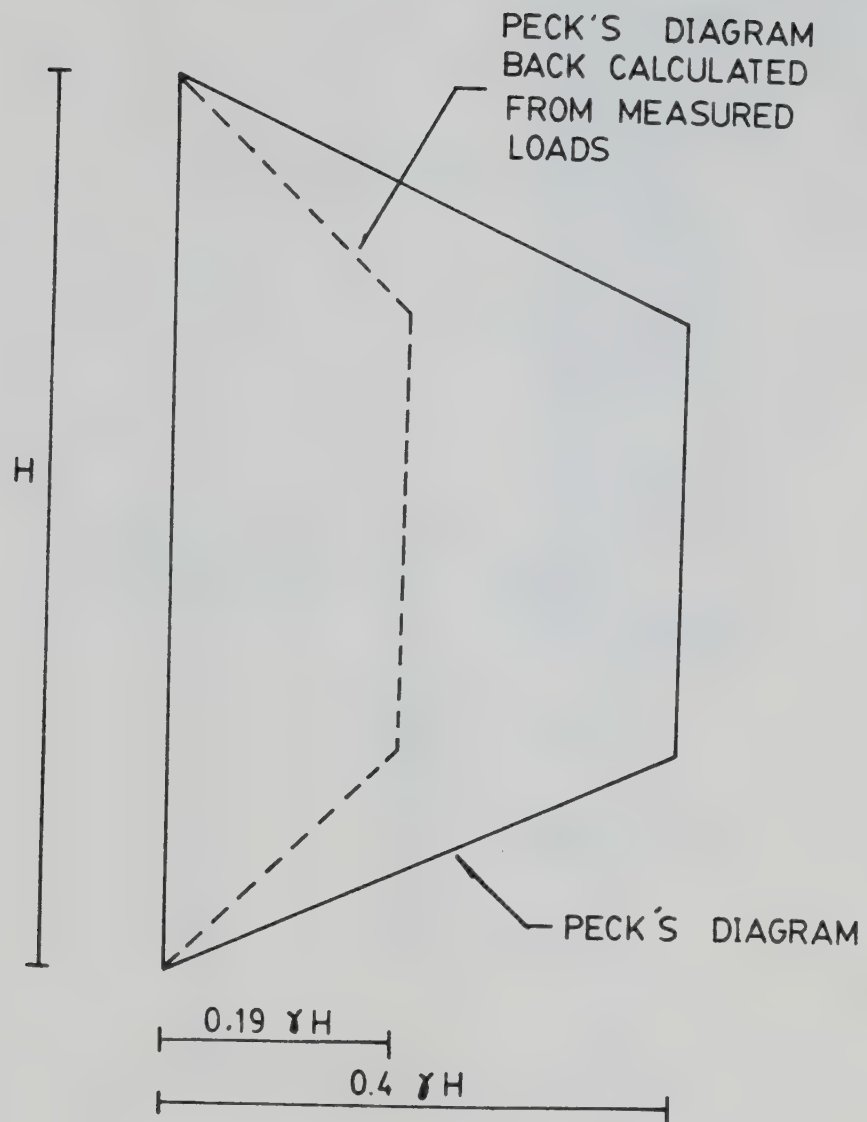


Figure 2.7 Pressure diagram resulting from measured load
(Sego, 1982)

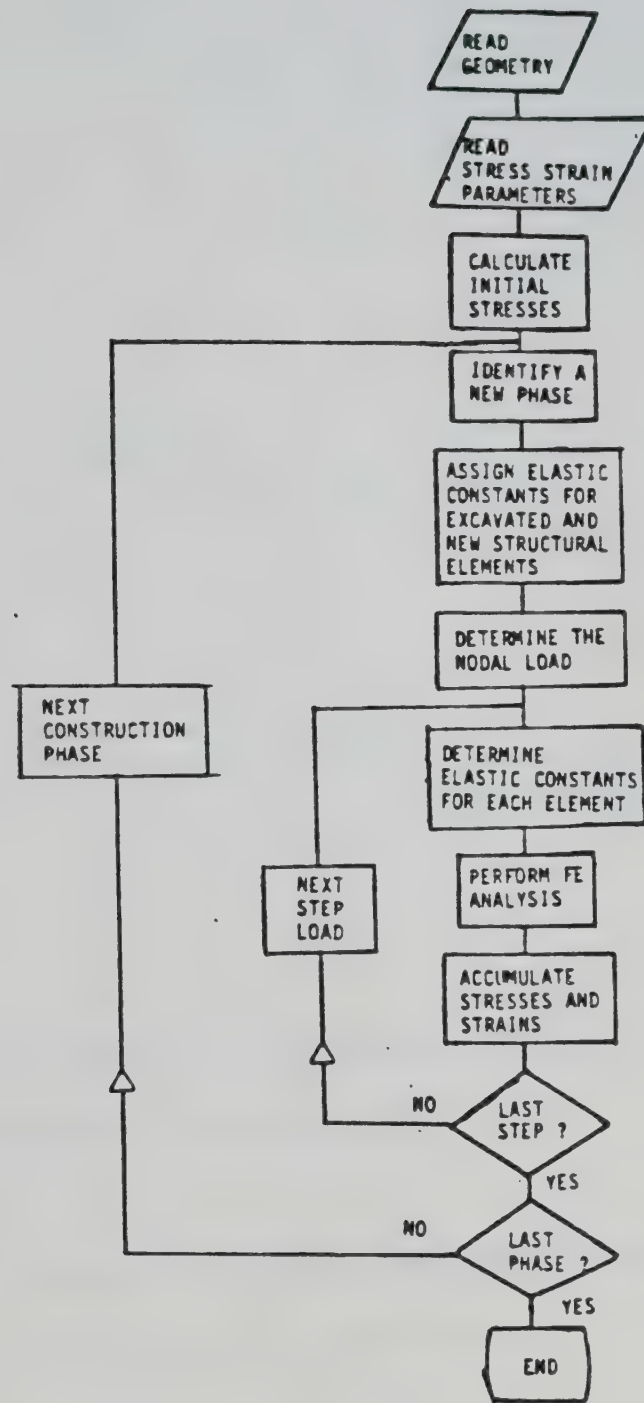
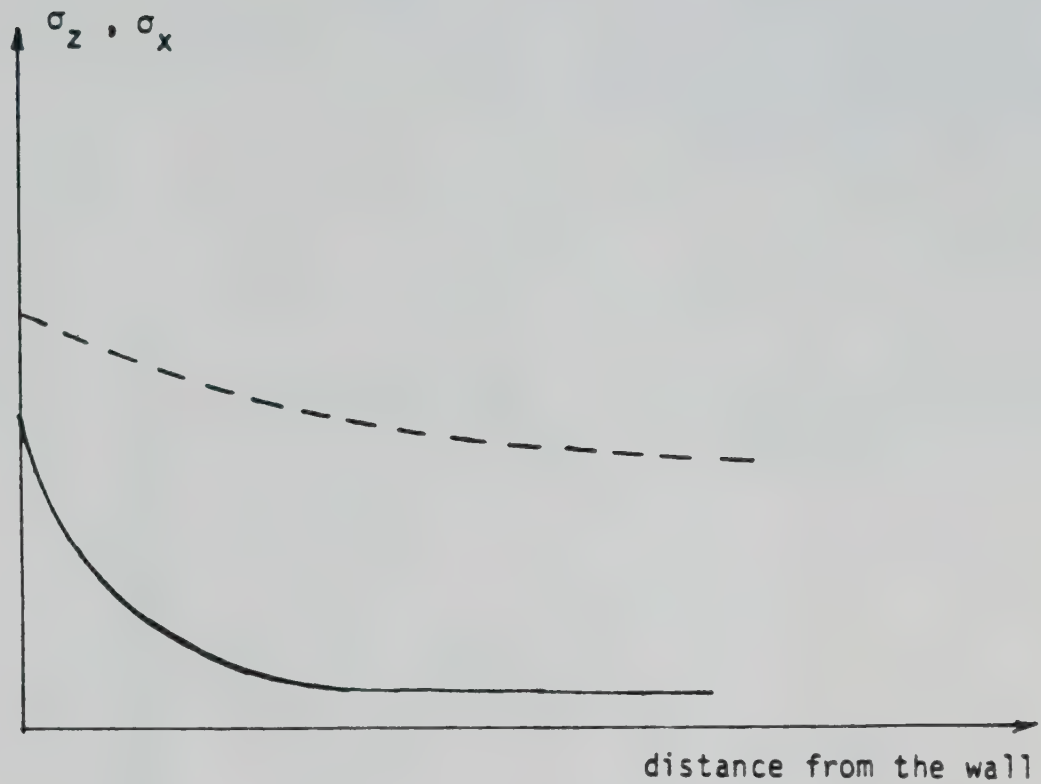


Figure 2.8 Logic of the program (After Medeiros, 1979)



———— horizontal line immediately below the
bottom of the excavation

----- horizontal line at some distance from
the bottom of the excavation

Figure 2.9 Stress distribution below the bottom of the excavation (After Medeiros, 1979)

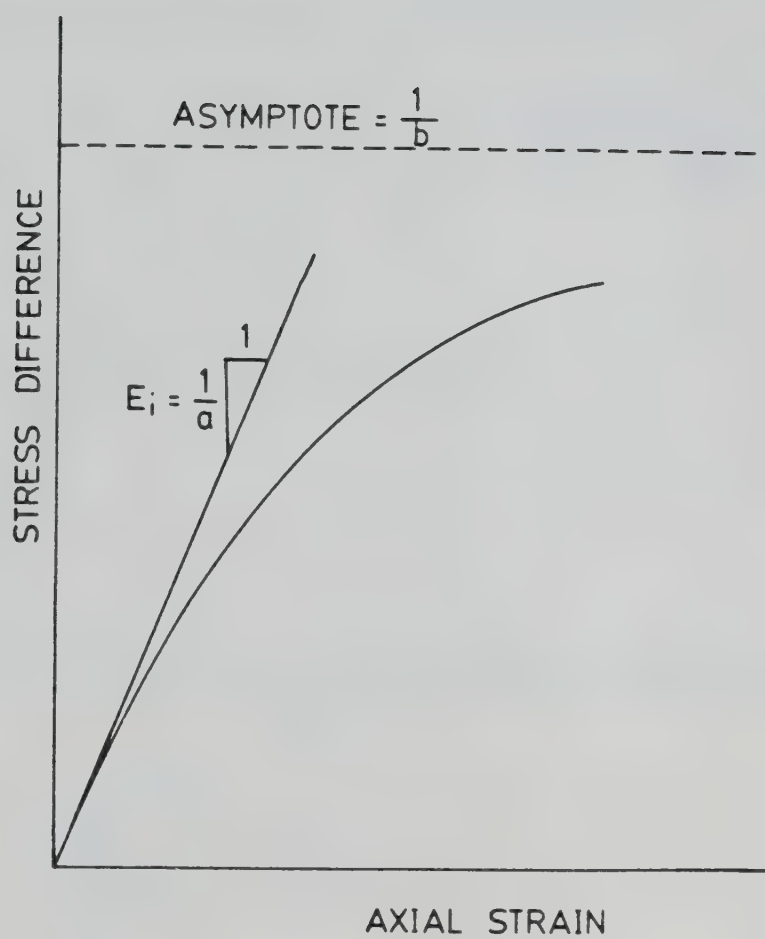


Figure 2.10 Hyperbolic stress strain curve

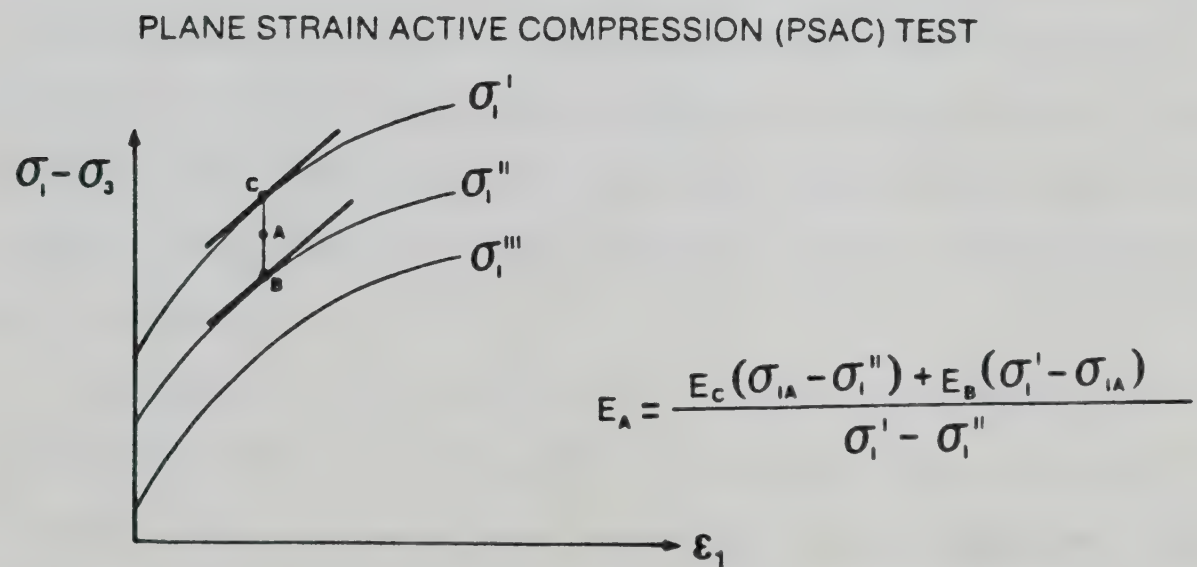
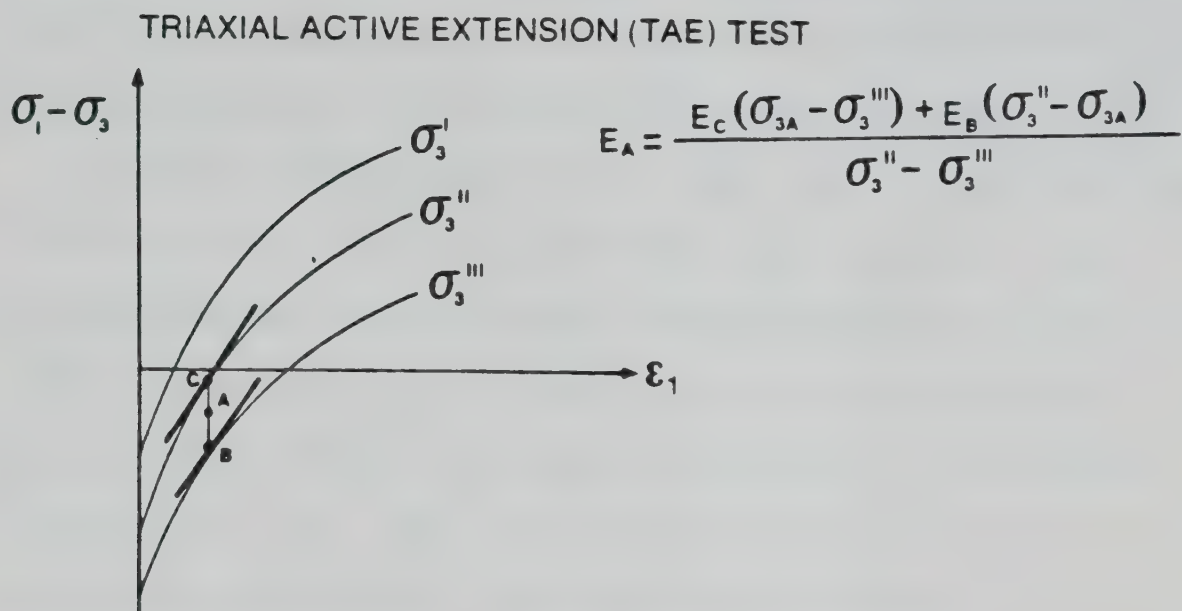
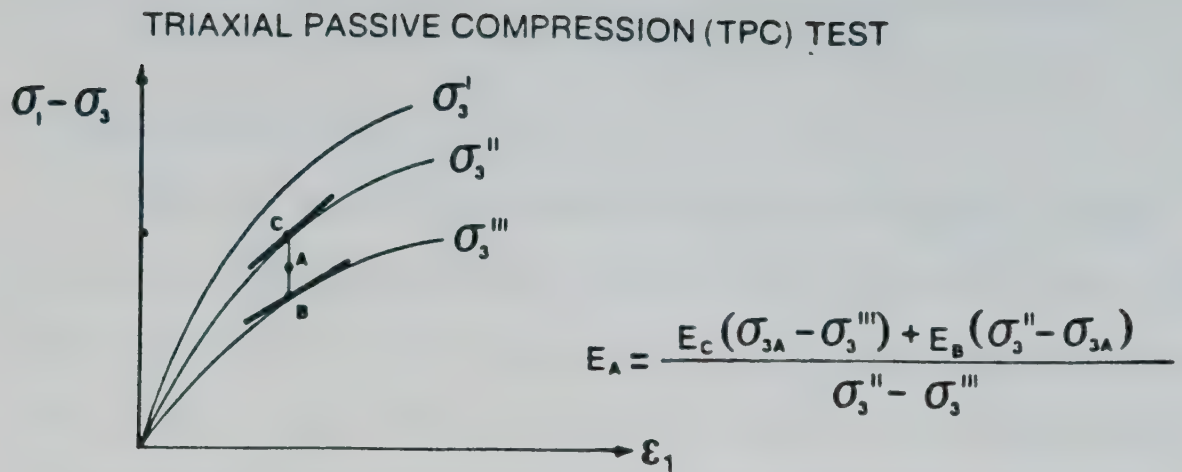


Figure 2.11 Modulus of deformation interpolated linearly between 2 curves (After Medeiros, 1979)

3.0 Description of the field cases studies

3.1 Introduction

Several field measurements have been made in the Edmonton area. Settlement records over nearly 6 years have been observed for two multistory buildings found on the clay till in Edmonton. The foundation for the CN tower was a spread footing and Franki expanded base concrete piles were used in the Avord Arms. A maximum of about 30 mm settlement was recorded on both building (De Jong and Harris, 1971).

Similar measurements were made for two tall building foundations in Edmonton. Footings were used for the foundation of the 34 storey Alberta Government Telephone (AGT) and the 28 storey Oxford building (De Jong and Morgenstern, 1973). Two and a half years of settlement record were made. The readings were taken from more than 100 basement column plugs. The maximum settlement of the Oxford building was about 28 mm whereas the AGT tower showed a maximum of 48 mm. The finite element predictions and field measurements were compared and the linear elastic method of calculation provided good agreement between the prediction and measurement.

Medeiros (1979) obtain field measurements for the Churchill Station of the LRT in Edmonton. Settlements, deflections and loads were recorded and analysed for the tangent pile supported excavation. Subsequently, similar measurements were made at the 104 Street Station and 107

Street Station which were reported by Hardy Associates (1982) and Thurber Consultants (1983) respectively. Field records of the deflections of the soldier pile and lagging wall and load measurements on tie back made by Sego (1982) are also available. Prior to presenting the field measurements on tie back and strutted excavation made in the Edmonton till, the local geology will be discussed. Then each project will be discussed as to construction procedure and its design assumption. The field measurements will be presented in a subsequent chapter.

3.2 Geology of the Edmonton area

Edmonton is located in the physiographic region known as the Eastern Alberta Plain (Alberta, 1969). It is an area of low relief, with elevations ranging from 700 meters to 830 meters.

During late Cretaceous time, the Edmonton area was covered with a shallow continental sea. The deposits of clay, silt and sand formed mudstones, clayshales and sandstones. After the uplift in the Cenozoic, the bedrock surface was eroded by a well integrated river system (Kathol and McPherson, 1975). The present North Saskatchewan River was formed by the last major erosion cycle. Portion of these channels were filled with late Tertiary sands and gravels known as the Saskatchewan sands and gravels.

Deposits of the Quaternary Periods in the Edmonton area consist of a lower and an upper till. The lower till,

rest on the Saskatchewan sands and gravels. It was laid down by ice moving from west to north. This till is greyish with a rectangular joint system. A later advancement of ice moving from east to north formed the upper till which has a brownish colour, with a columnar system of joints. In some areas, the till contains pockets of sands that vary in size from one to several metres in lateral dimension and from several centimetres to a metre thick. These sand lenses often are water bearing and the water may be under pressure.

Above the upper till are silty clays, deposited in glacial Lake Edmonton.

3.3 Local profile

Some boreholes were drilled along the LRT route. Test data obtained and a brief summary of the soil profile and description is contained in table 3.1 and figure 3.1. Detail borehole profiles are illustrated in appendix C.

As shown in figure 3.1 and table 3.1, above Lake Edmonton clay is recent fill which consists of asphalt, concrete and gravel pavement. In between the Lake Edmonton clay and Saskatchewan sands are the lower and upper till.

During the construction of the 104th Street Station and the 107th Street Station, water bearing sand lenses were intersected between 12 to 17 metres below street level.

3.3.1 LRT Stations

The Edmonton Light Rail Transit System (LRT) was proposed and constructed due to the rapid development of the city of Edmonton. Figure 3.2 shows various underground stations located in the city core.

Between the Churchill and Central stations, two parallel tunnels were constructed and their performance has been described by Eisenstein and Thomson (1978). The 104 Street and 107 Street Stations were constructed by cut and cover construction as had the Churchill and Central Station. The station wall consisted of cast in place concrete tangent piles. The 104 Street Station, 107 Street Station and the Churchill Station were selected for analysis and field measurements of deformations. It should be noted that the Churchill Station was analysed and the findings described by Medeiros (1979), and Medeiros and Eisenstein (1983).

3.3.2 106 Street and 100 Avenue excavation

A temporary shoring system consisting of steel soldier pile and timber lagging between the soldier piles was used for the excavation support of two and one half levels of underground parking beneath a new office tower located at the corner of 106 Street and 100 Avenue. The site was chosen for investigation because the retaining structure was more flexible than the concrete tangent pile wall previously studied at the LRT stations and reported by Eisenstein and Medeiros (1983). Figure 3.2 illustrates the

location of the construction site.

3.4 Structure and construction Procedures

3.4.1 LRT Stations

The retaining structures for the excavation at the 104 Street Station and 107 Street Station are composed of concrete tangent pile walls. The tangent piles were installed to a depth of about 21 meters below existing grade with a diameter of 1067 mm. Figure 3.3 illustrates a typical cross section of the station. The wall was laterally supported by struts at the street level, mezzanine level and at the base of the station. Figure 3.4 shows the cross sections of the typical 'L' shape beam and the precast girder.

The 'L' shape concrete beams were placed at the top of the tangent piles which runs parallel to the axis of the excavation. The precast girders were supported by this 'L' shape beam as shown in figure 3.5. Both mezzanine floor and bottom slab were cast in place concrete structures. Figure 3.6 shows the geometry of the floor strut.

The construction sequence starts with drilling holes for the tangent piles. The holes are then poured with concrete after the placement of reinforcement cage. When all the tangent piles are casted, excavation begins. The 'L' shape beam are cast on top of the tangent piles after a small excavation. The precast girders are then placed on the

'L' beams after the depth of the excavation is deep enough to provide space for the excavation equipment to work below the precast beams. Excavation proceeds and the mezzanine floor, the second level strut, is cast in place. Finally, the excavation reaches the designed depth, the bottom floor slab concrete is then placed.

The construction of the Churchill Station has been described by Medeiros (1979), Esienstein and Medeiros (1983) and therefore it is not necessary to elaborate on its construction.

3.4.2 106 Street and 100 Avenue excavation

The building site provided sufficient clearance to allow an unretained earth slope to a depth of 5.2 meters. H piles were then driven from the excavation into the soil to a depth of 14.05 meters from the ground surface with a spacing of 2471 mm. As the excavation proceeded to the depth of about 3 meters below the top of the H pile, sloping ground anchors were drilled and cast. At the same time, a whaler was constructed between the soldier piles to transfer the load via high strength steel anchor rods which had previously been constructed. The diameter of the ground anchor concrete grout ranged from 300 to 400 mm. The final depth of the excavation was 5.8 meters below the top of the H piles. Figure 3.6 illustrates the cross section with the soil profile.

3.5 Lateral design pressure for each walls

3.5.1 LRT tangent pile wall

The tangent pile wall were braced as rigidly as possible to minimize ground movement. Figure 3.8 contains the design pressure distribution as recommended by Thurber Consultants for both the 104 Street and 107 Street Stations. A linear increase of pressure with depth from street level to the mezzanine floor with a magnitude of $18.3D$ was proposed, where D is the depth between the street level and mezzanine floor. Below the mezzanine floor, a constant pressure distribution of $18.3D$ was recommended. The passive resistance from the embedded portion of the tangent piles was triangular in shape with a magnitude of $18.3P$, where P is the depth of penetration of the tangent pile below the final excavation depth.

3.5.2 106 Street and 100 Avenue solidier pile wall

A magnitude of $0.4\gamma H$ suggested by Peck (1969) for stiff soil was used in the design pressure diagram. Instead of a trapezoidal shape pressure distribution, the lower portion of the pressure diagram was increased to a uniform pressure as shown in figure 3.9. Detail description of various earth pressure diagrams have previously been discussed in Chapter 2.

MATERIAL	DESCRIPTION	DEPTH (M)
FILL	ASPHALT OVER CONCRETE COMPACT BROWN SANDY GRAVEL	0 TO 1.0
LAKE EDMONTON CLAY	VERY STIFF BROWN SILTY CLAY NUGGETTY STRUCTURE WITH WHITE SALTS GASEOUS ODOURS	1.0 TO 5.5
UPPER TILL	HARD BROWN SILTY SANDY CLAY WITH SAND SEAMS, PEBBLES, COBBLES AND COAL FRAGMENTS	5.5 TO 13.
SAND LENS	DENSE WET SAND COBBLES FOR 0.3 METER	13 TO 15.5
LOWER TILL	HARD GREY SILTY SANDY CLAY WITH PEBBLES COAL CHIPS AND NUMEROUS WET SAND SEAMS	15.5 TO 26
SASKATCHEWAN SANDS	VERY DENSE BROWN FINE GRAINED UNIFORM SAND	26 TO ?
EDMONTON FORMATION	INTERBEDDED MUDSTONE AND SILTSTONE	?

Table 3.1 Soil profile

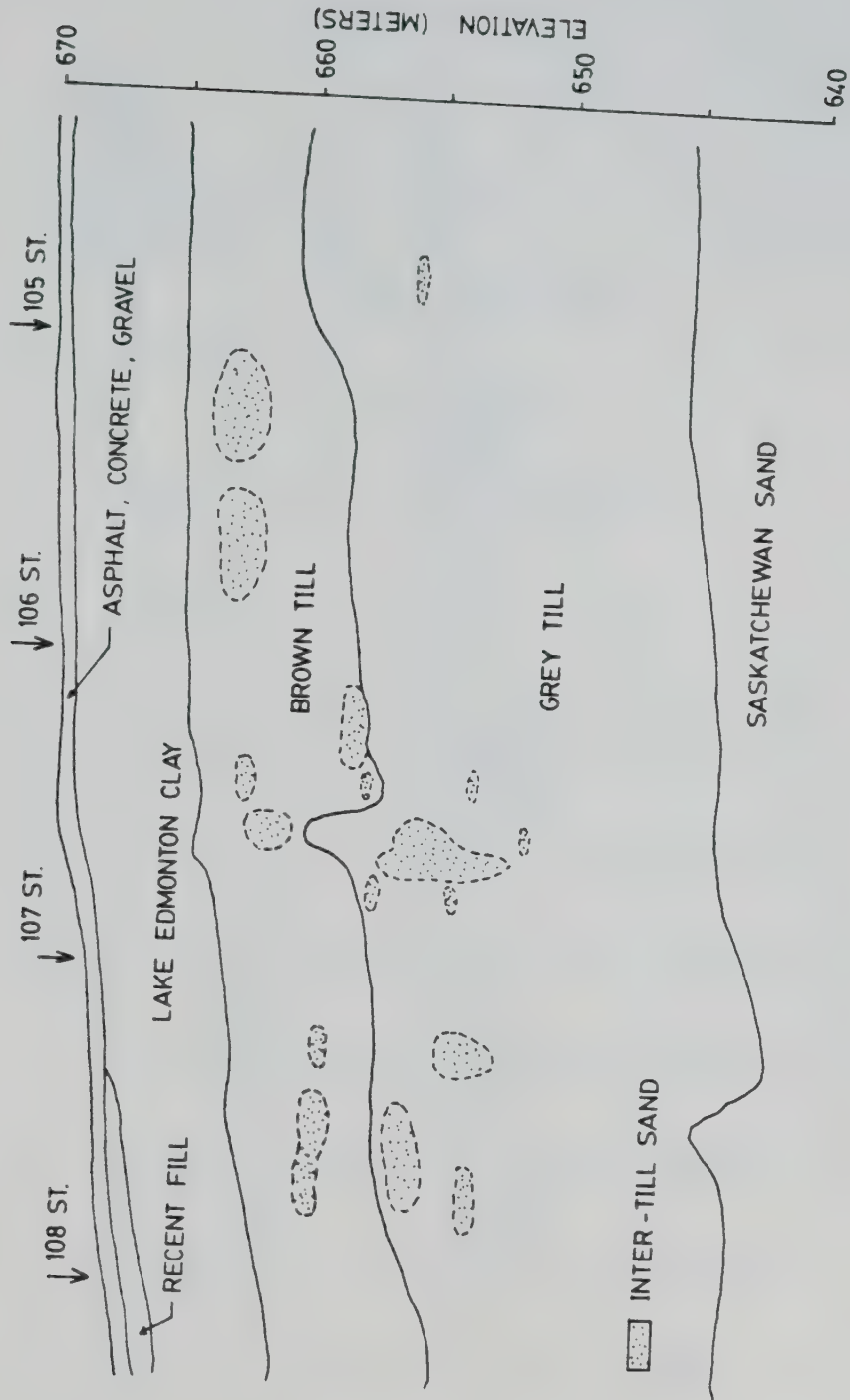


Figure 3.1 Soil profile

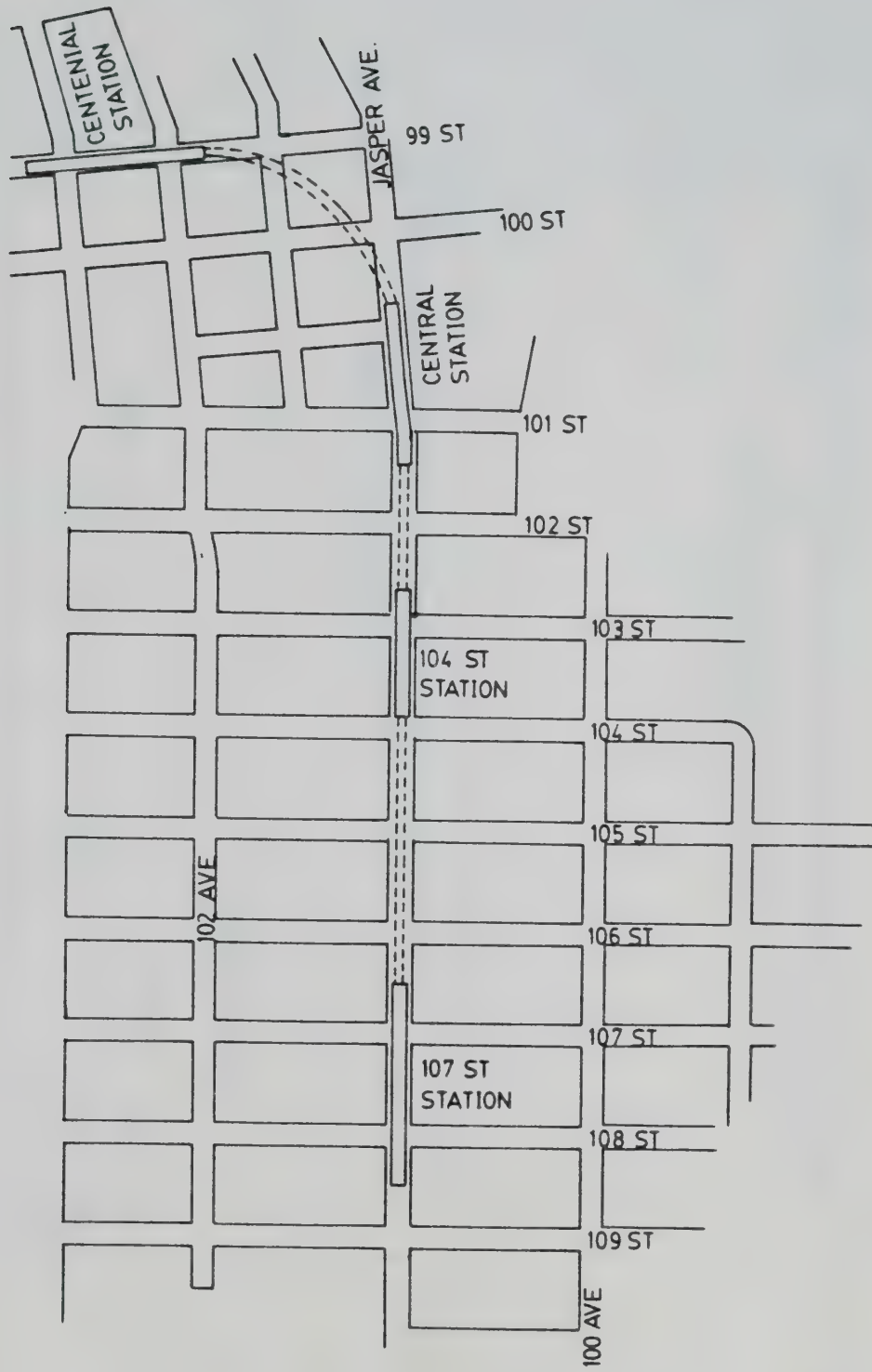


Figure 3.2 Site plan showing locations of downtown LRT stations (Thurber Consultants, 1980)

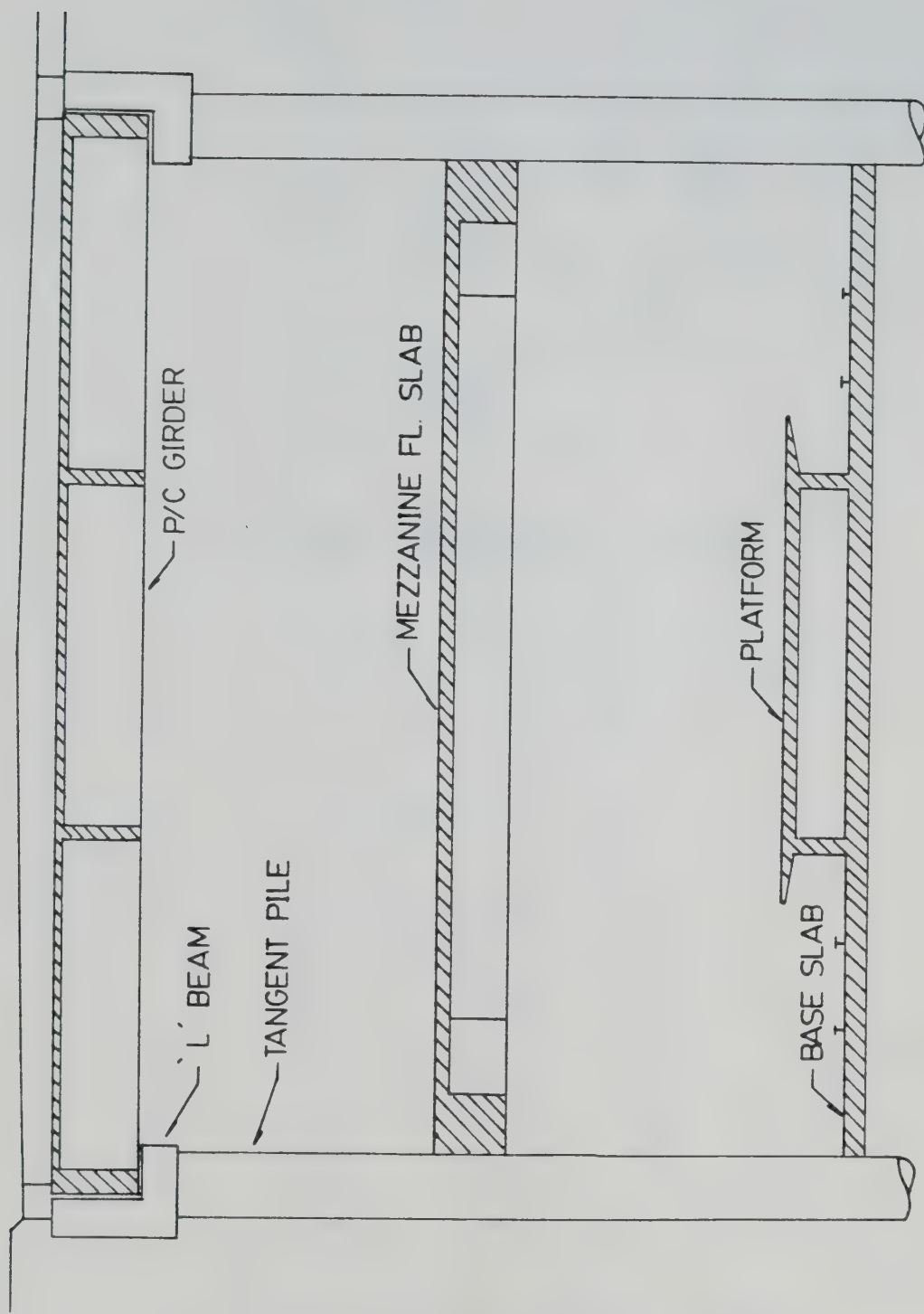
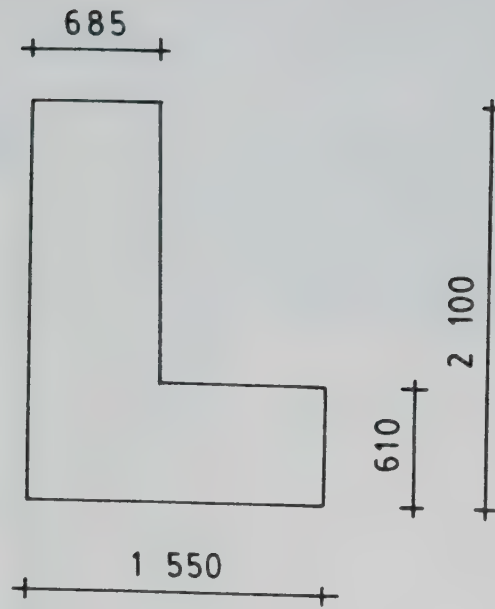
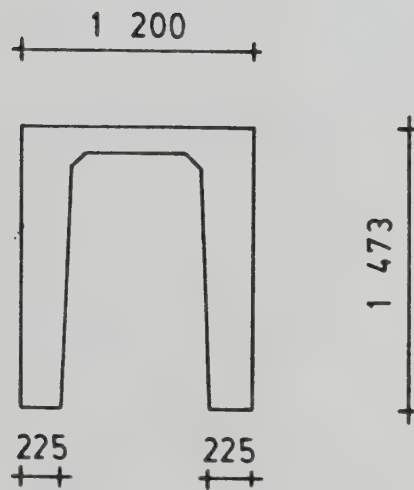


Figure 3.3 Cross section of the LRT station (Thurber Consultants, 1980)



CROSS SECTION OF 'L' BEAM



CROSS SECTION OF PRECAST GIRDER

Figure 3.4 Cross section of L beam and precast girder
(Thurber Consultants, 1980)

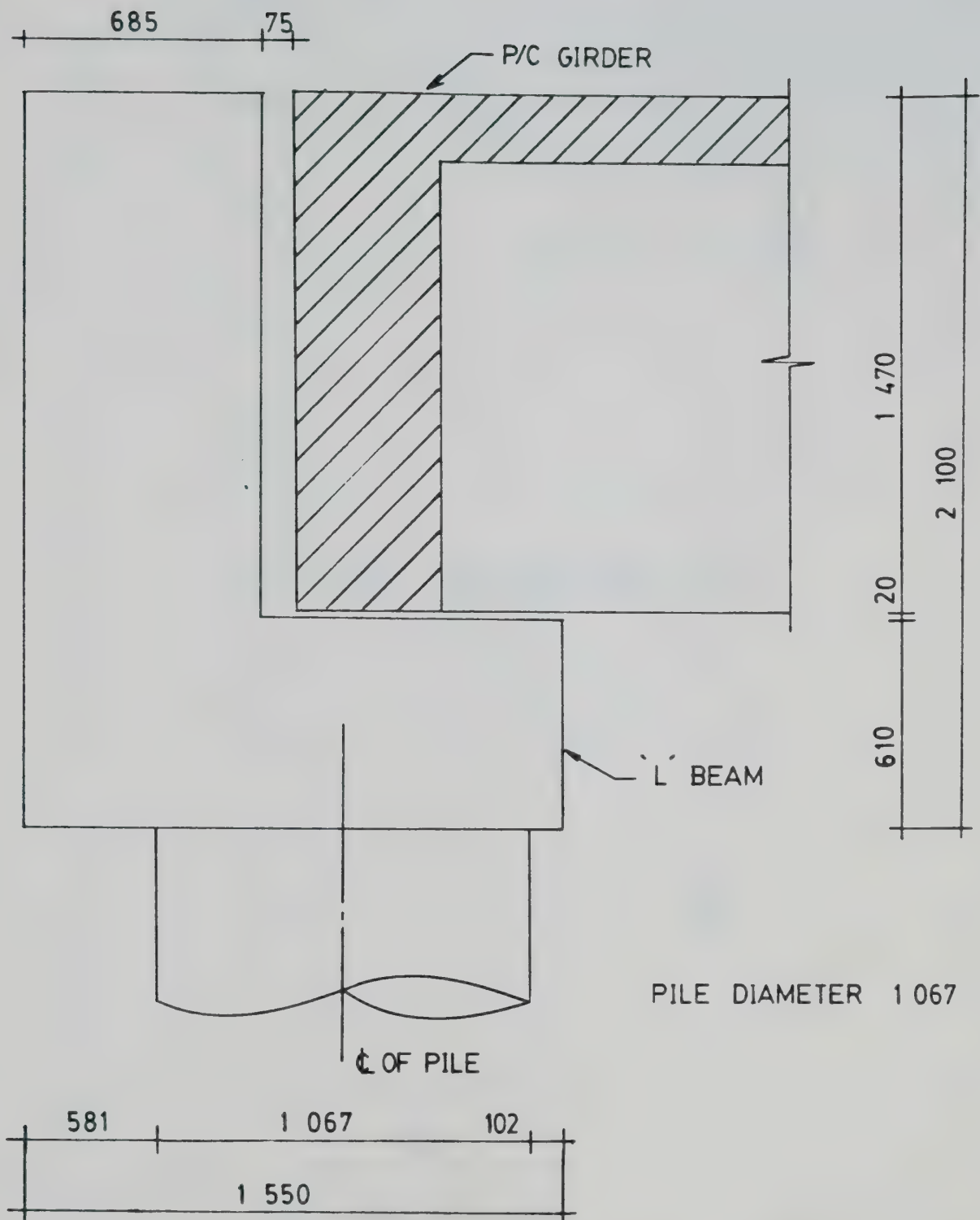
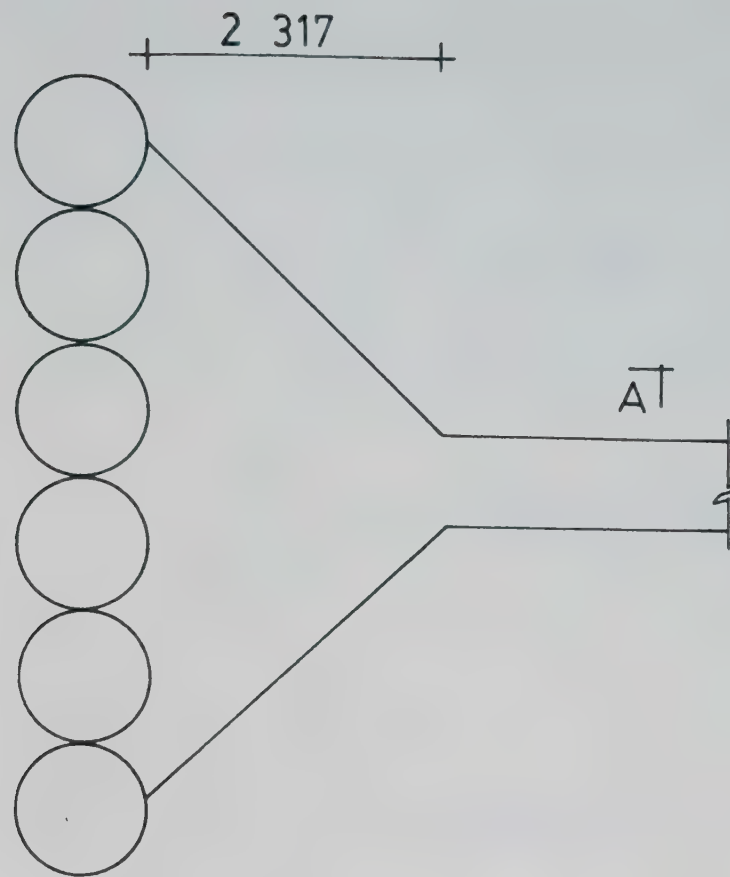
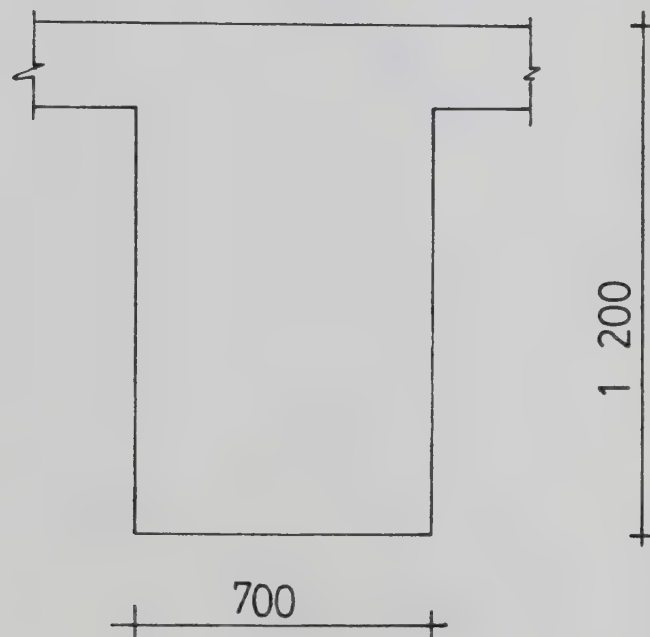


Figure 3.5 Connection between L beam and P/C girder (Thurber Consultants, 1980)



PLAN VIEW OF MEZZANINE BEAM



SECTION A-A

Figure 3.6 Plan and section of mezzanine beam (Thurber Consultants, 1980)

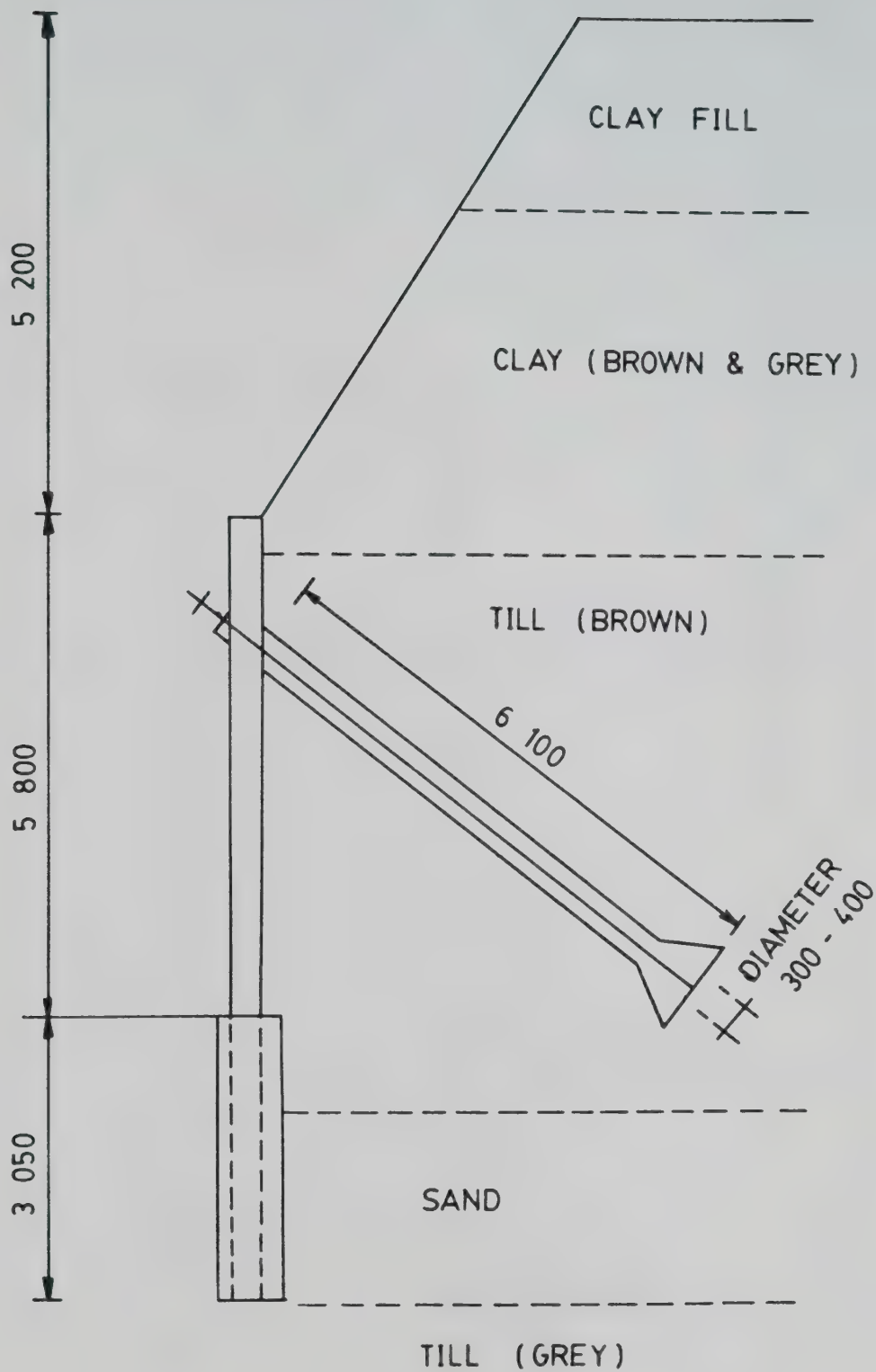


Figure 3.7 Cross section through the H pile wall at 106 Street and 100 Avenue (Sego, 1982)

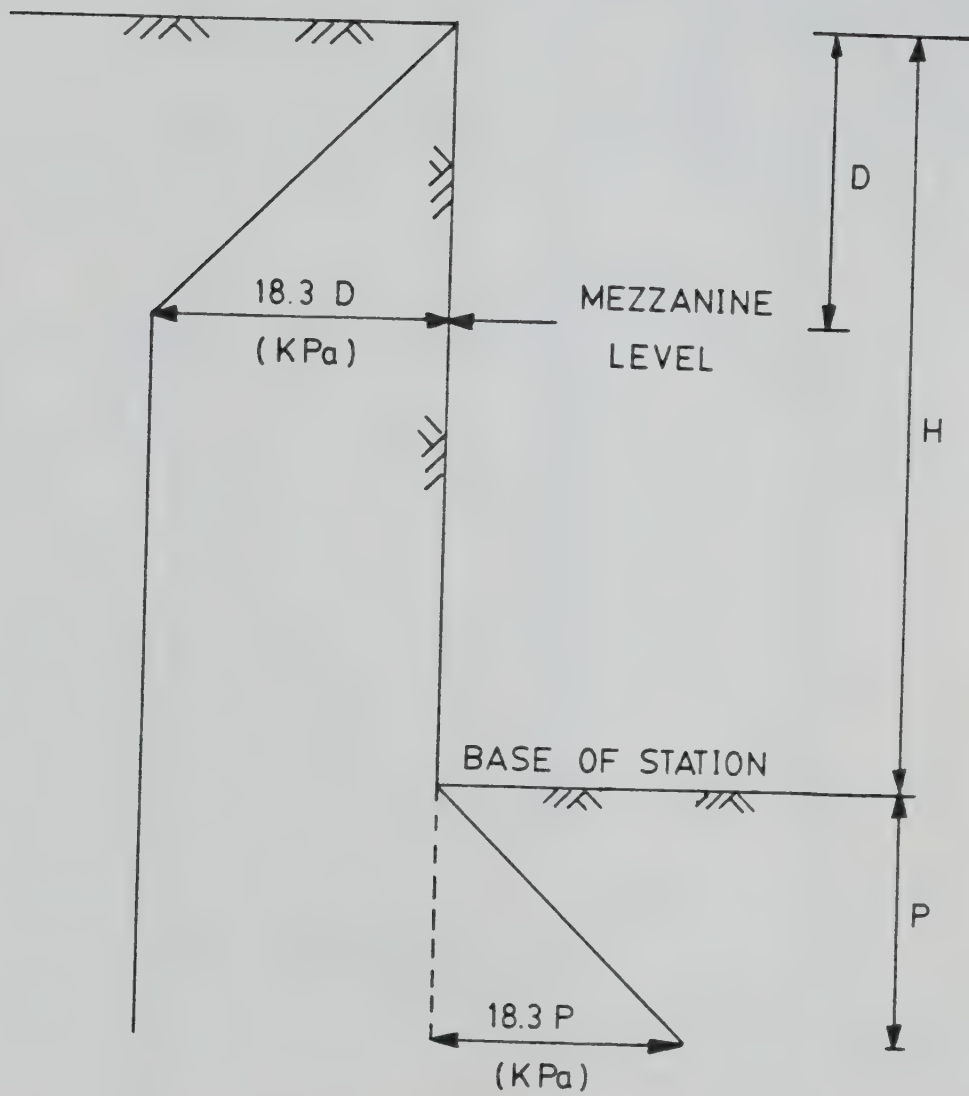


Figure 3.8 Design pressure recommended by Thurber Consultant

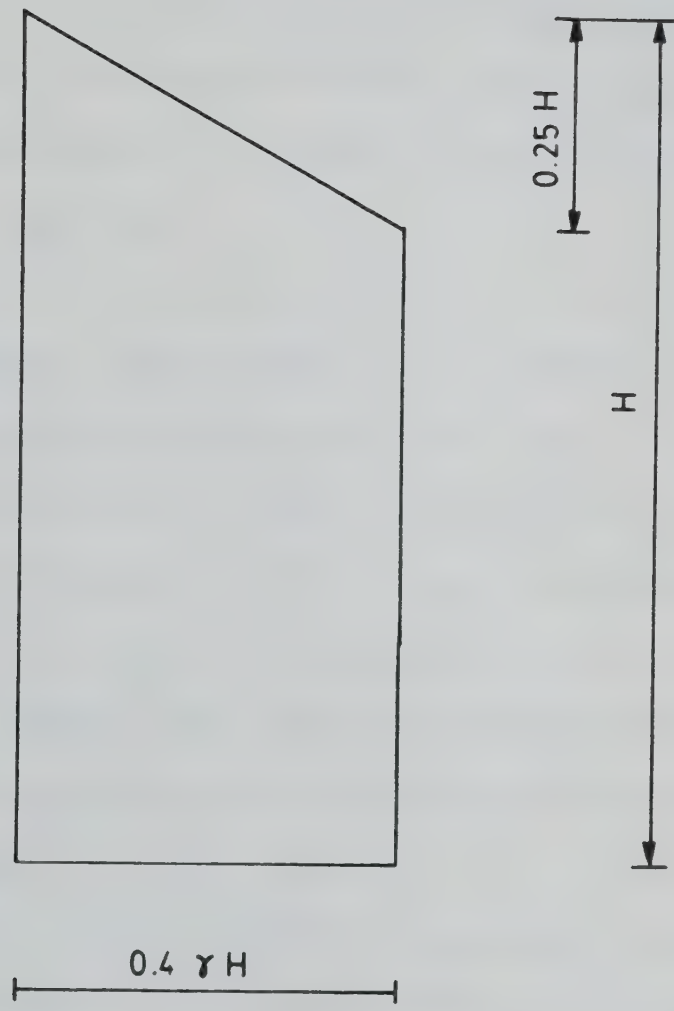


Figure 3.9 Design pressure for the H pile wall (Sego, 1982)

4.0 Description of field measurements

4.1 LRT stations

The layout of the instrumentation of the Churchill station was described by Medeiros (1979) and shown in figure 4.1. A detail discussion and description of it can be found in his thesis.

Figure 4.2 shows the layout of the instrumentation program at the 107 Street Station. Only slope indicators were installed at the 104 Street Station and figure 4.3 shows their locations.

4.1.1 Multipoint extensometer at 107 Street Station

Before starting the excavation, three magnetic multipoint extensometers MPX1 to MPX3 were installed inside the 107 Street Station. The vertical movements of the soil were measured by these instruments during the excavation between the street level and the mezzanine floor. The instruments were destroyed during excavation beneath the mezzanine slab.

The magnetic multipoint extensometer consists of a ring of magnets installed in a hollow PVC cyclinder. A borehole was drilled and a smaller diameter PVC pipe was installed. Bentonitic slurry was back filled into the borehole in order to prevent sloughing of the borehole wall. Several magnets could be pushed over the PVC pipe to the desired elevation for future measurements. Attached to the outside wall of the

PVC cyclinder were metal spring legs which provide the secure contact between the soil and the PVC magnet holder. Therefore the cyclinder which contains the magnets moves with the soil relative to the inner PVC pipe. The movement of the magnets was measured by a special sensor which detects the magnetic field surrounding each magnet. In this manner the relative soil movement was recorded at various depths.

The readings of the multipoint extensometers as the excavation proceeded to the mezzanine level are presented in figure 4.4, 4.5 and 4.6. The maximum heave recorded was about 6 mm. It can be observed that settlement occurred just below the bottom of the excavation or base floor and heave occurred in the soil above this elevation.

4.1.2 Settlement points

As shown in figure 4.2, 5 settlement points SP1 to SP5 were installed along the west sidewalk of 108 Street, and 5 settlement points SP6 to SP10 were installed in the middle of 107 Street. The settlement points were placed approximately 1.5 meter below the ground level. During the excavation of the construction entrance at 107 Street. SP6 to SP10 were damaged and no readings were recorded during the project.

The measurements of the settlement points SP1 to SP5 are presented in figure 4.7 and 4.8 for the excavation to the mezzanine level and for the excavation between the

mezzanine level and the base slab respectively. A maximum heave of 2.5 mm at SP1 and settlement of 2.5 mm at SP2 were recorded when excavation reached the mezzanine level. After about 6 weeks of casting the base slab, a reading of about 4.5 mm settlement was recorded. It is of practical interest that the settlement point (SP5) located at about 20 meters north of the tangent pile wall was unaffected by the excavation in the station.

4.1.3 Slope indicator

Slope indicators were installed at both 107 Street Station and 104 Street Station to monitor the horizontal movements of the tangent piles.

1. 107 Street Station

Four slope indicators were installed at 107 Street Station as shown in figure 4.2. Slope indicators SI1 to SI3 were embedded in the tangent piles and SI4 was embedded in the tangent pile which had a steel column from street level to mezzanine floor to measure the deflection of the shoring system beneath the beam. Figure 4.9, 4.10 and 4.11 show the deflection at various stages of excavation as recorded by slope indicators. The maximum movement recorded at the end of construction was in the order of 4 to 6 mm.

2. 104 Street Station

Four tangent piles were instrumented with slope indicators at the 104 Street Station with two on the south wall and the other two on the north wall as shown in figure

4.3. Slope indicator 103 was damaged during construction but it was repaired and serviceable. The readings recorded by slope indicator 243 were adjusted since the slope indicator was rotated during installation. The maximum horizontal movements recorded varied between 5 and 8.5 mm as shown in figures 4.12 to 4.15. It should be noted that pile 103 was already laterally loaded before any reading was taken. Therefore slope indicator 103 does not show the actual deflection due to the excavation of the station but illustrates the horizontal movement since October 30, 1981.

4.1.4 Lateral loads and stresses at 107 Street Station

1. Strain gauges

Weldable vibrating wire strain gauges and embedded vibrating wire strain gauges were used. Eight weldable strains gauges were spot welded on the flange of the steel H-pile embedded in the concrete pile at the connection between the station and the NATM tunnels. The strain gauges were spot welded near the elevation of the center of the tunnel. Six embedded strain gauges were installed in the mezzanine floor beam and the remaining were embedded at the mid-height of the slab. The purpose of installing these strain gauges were to measure the loads resisted by the mezzanine level strut.

The strain gauge measurements after the excavation of the station are shown in figure 4.16. A value of 407.5 microstrain or 8.5 MPa was recorded on September, 1982 and

an average strain of 463.2 microstrain or 9.7 MPa was recorded on May, 1983. A measurement indicated 265 and 241 microstrain in the mezzanine floor corresponds to an average compressive stress of 5.3 MPa was recorded during May, 1983.

The weldable strain gauges showed the strain and stress distribution in figure 4.17 and figure 4.18. Considered as a simply supported beam with uniform load, the stress distributions between 35 and 65 MPa was estimated from the measurements.

2. Load cells

In order to measure the lateral load at the street level due to excavation, two load cells were placed at the contact between the precast girder and the vertical leg of the L beam. The load cells were cyclindrical in shape with a diameter of 300 mm and 75 mm high.

Readings were taken at various stages of excavation. Before excavation to mezzanine level, the two load cells #1 and #2 recorded 5.5 and 16.6 kN respectively. The loads increased to 33 and 45 kN when the excavation reached the mezzanine level indicating that a net load of 27.5 and 28.4 kN resulted from the excavation. Load cell #1 malfunctioned after casting of the mezzanine level, load cell #2 recorded an increase from 45 to 62 kN and drop to 14.5 kN when the excavation reached the final depth.

4.2 106 Street and 100 Avenue excavation

Figure 4.19 illustrates the layout of instrumentations. The load cells were used to measure the tie back loads and the movement of the wall was monitored by means of slope indicators. Dywidag bar of 32 mm was used as the tie back anchor. The slope indicator was attached to the soldier pile and thus measures the soil movement behind the soldier pile and not the movement behind the lagging.

4.2.1 Slope indicator and vertical movement

The slope indicator tubing was attached to the steel soldier pile using metal clips attached at the slope indicator couplers and welded to the flange of the H-pile. The system was then placed in a pre-drilled pile hole and the bottom 2 meters was filled with concrete which had a compressive strength of 25 MPa. A weak mix concrete with a strength of 2.5 MPa was placed above the 25 MPa concrete.

Five slope indicators were installed as shown in figure 4.19, four of them were placed at the north wall adjacent to 100 avenue and the remaining one installed on the south wall. Figure 4.20 to 4.22 shows a maximum deflection of about 4 mm from the measurements of slope indicator #7, #8 and #9. A maximum deflection of 3 mm was recorded by slope indicator #10 which was shown in figure 4.23. The record of the slope indicator on the south wall was missing in the report (Sego, 1982) and therefore could not be present here. The measurements were taken after the installation of the

tie backs. Different curves showed the different stages of the excavation and the last curve was recorded after the excavation was completed.

4.2.2 Load cell measurements

Three load cells were installed on the north wall adjacent to the instrumented section as shown in figure 4.19. Figure 4.24 illustrates the measurements. It can be observed that the measured load decreased in the first 6 days and then increased. After the tenth day, the record showed no significant change in the loads acting on the wall.

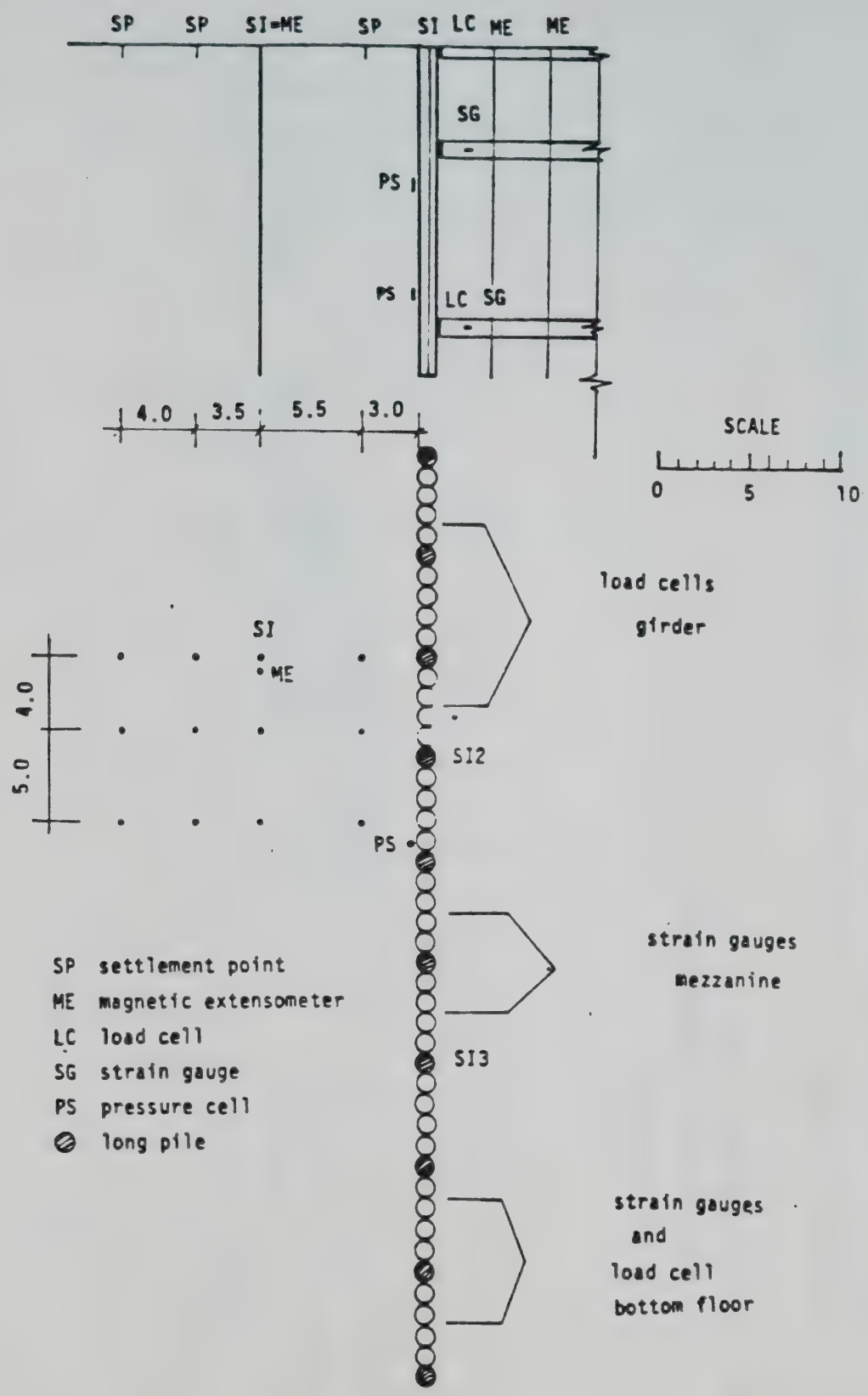


Figure 4.1 Layout of the instrumentation of the Churchill Station (After Medeiros, 1979)

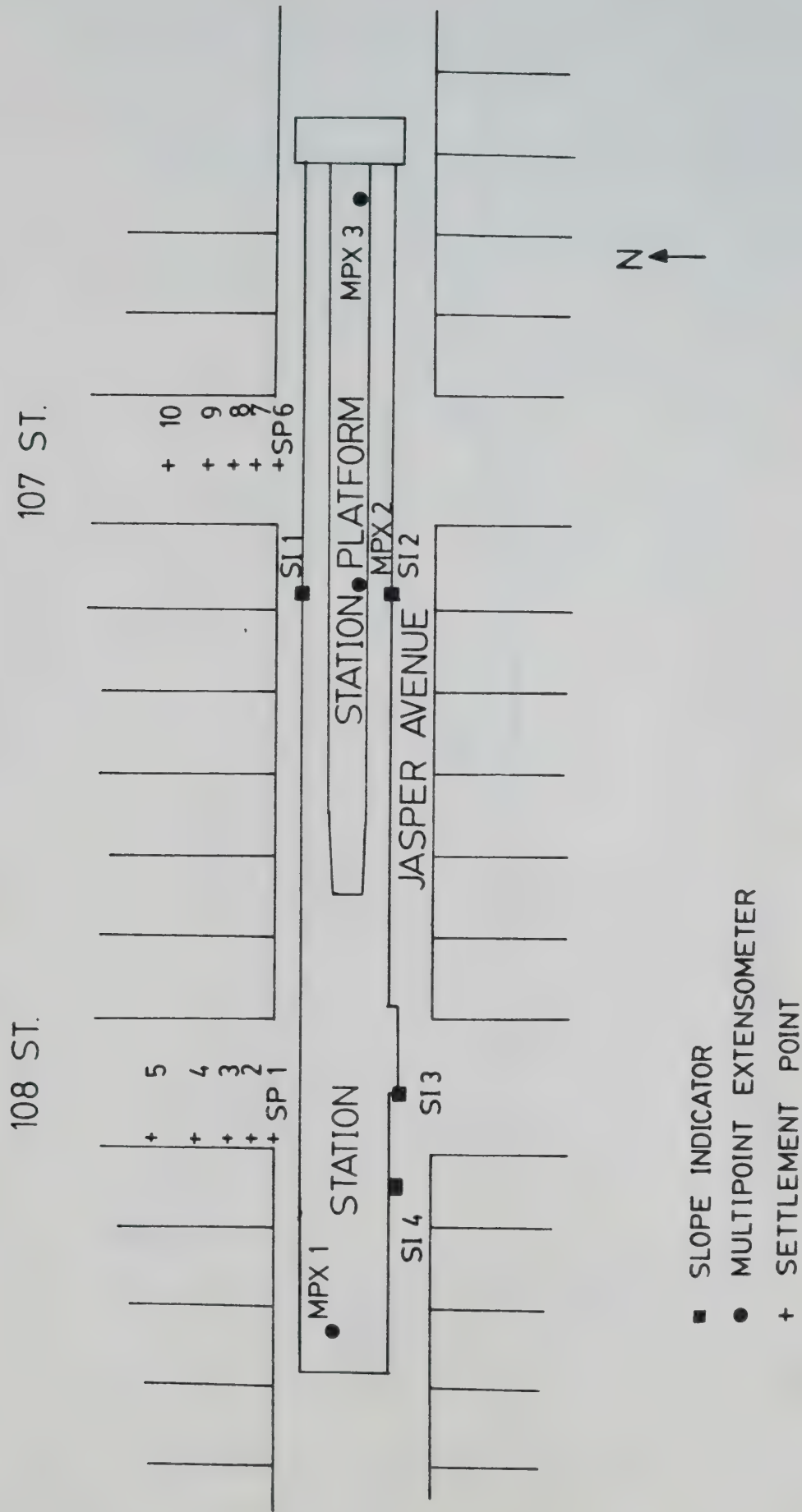


Figure 4.2 Layout of the instrumentation of the 107 Street Station (Thurber Consultants, 1980)

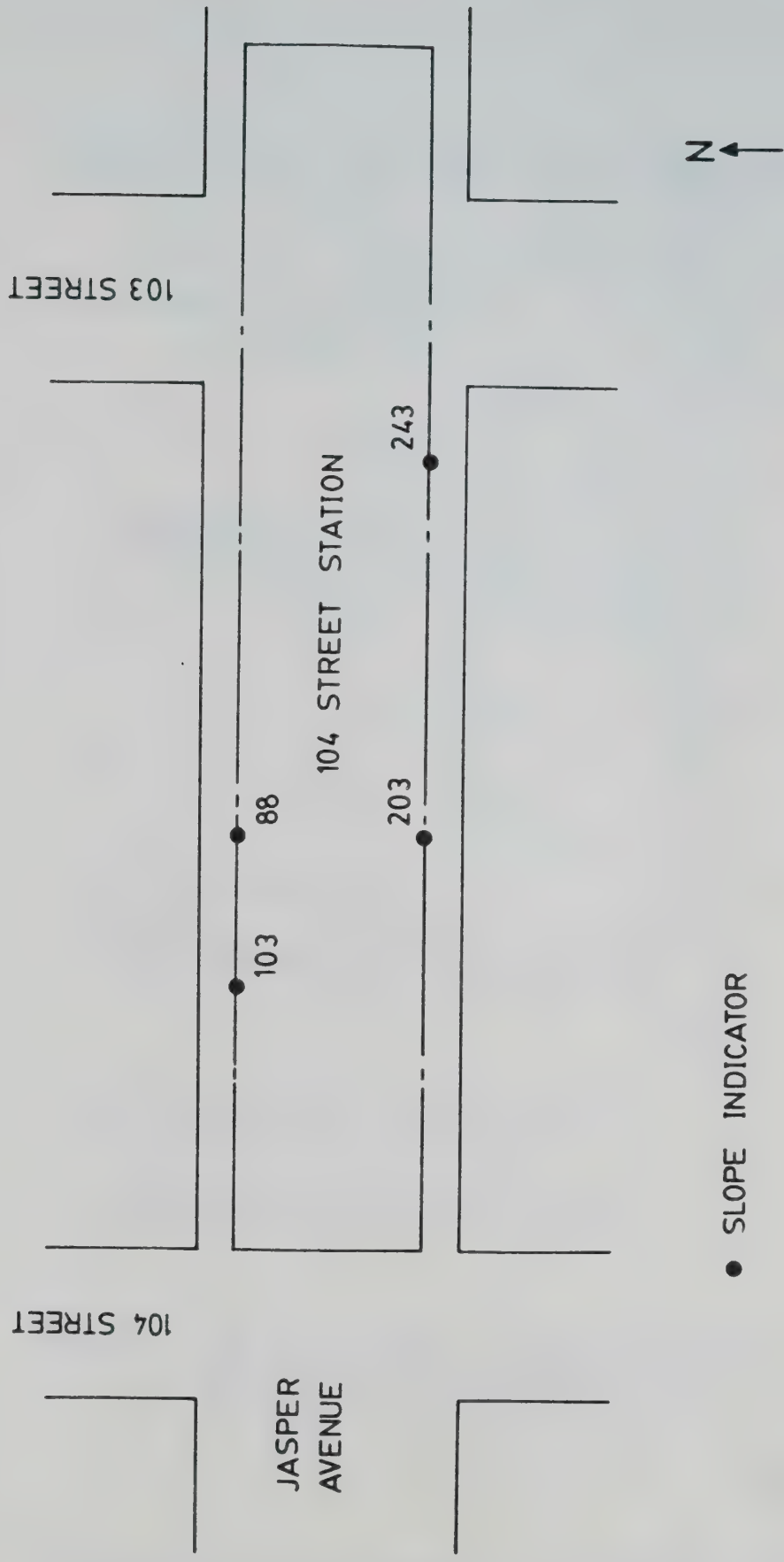
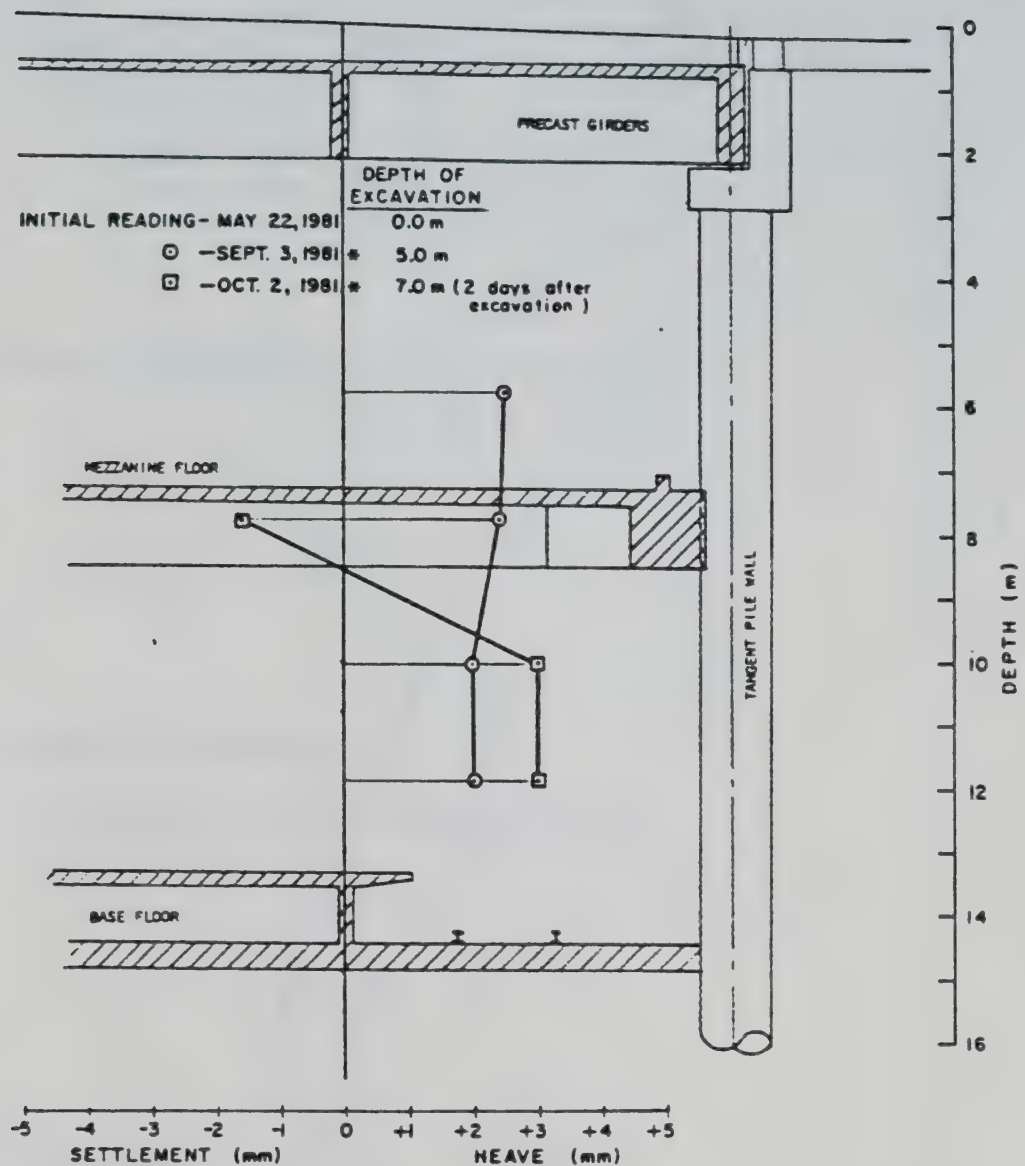


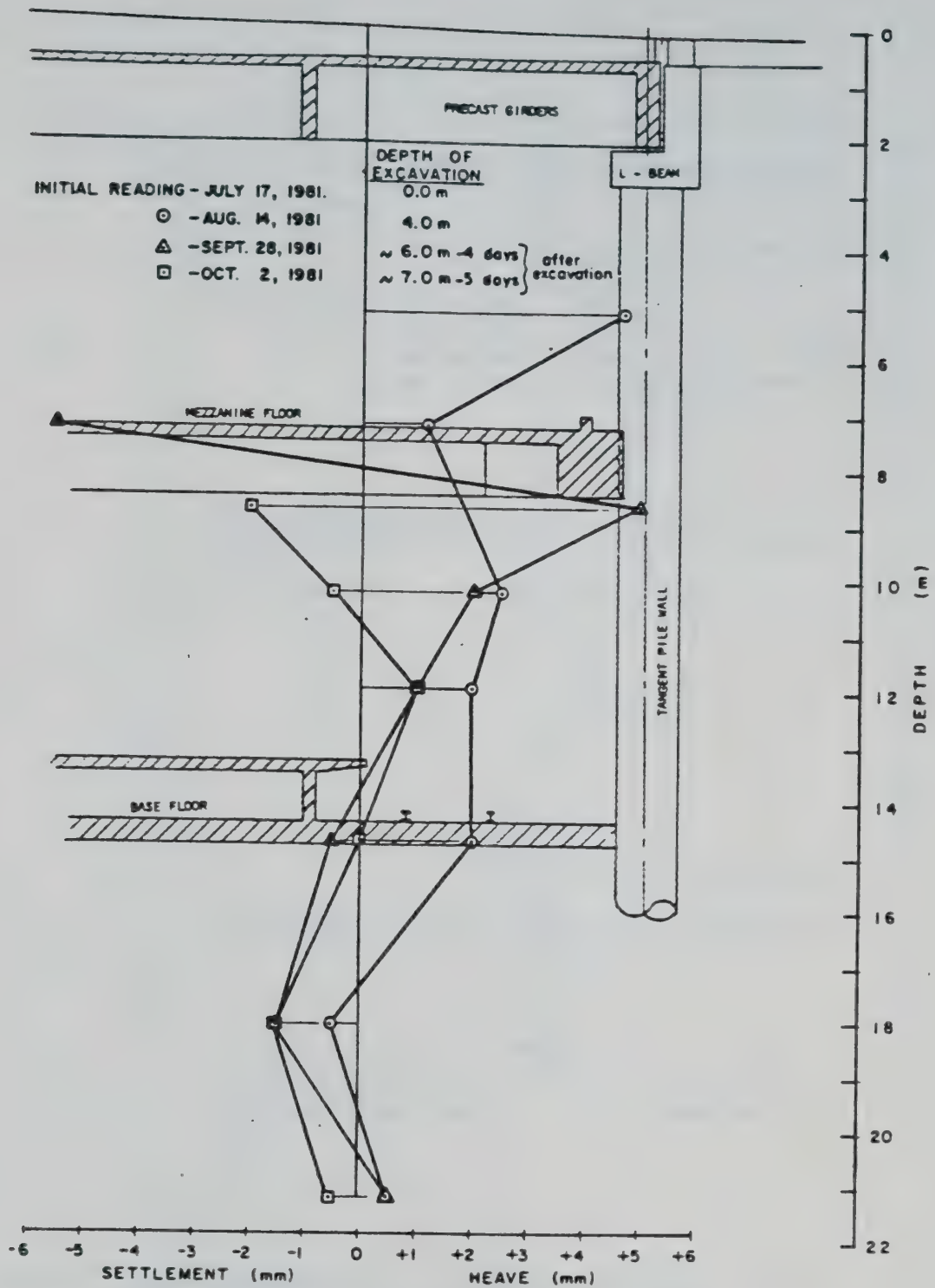
Figure 4.3 Layout of the instrumentation of the 104 Street Station (Hardy Associates, 1982)



* ASSUMED AFTER CORRECTION OF READINGS DUE TO DISTURBANCE OF MULTIPOINT AFTER INITIAL READING

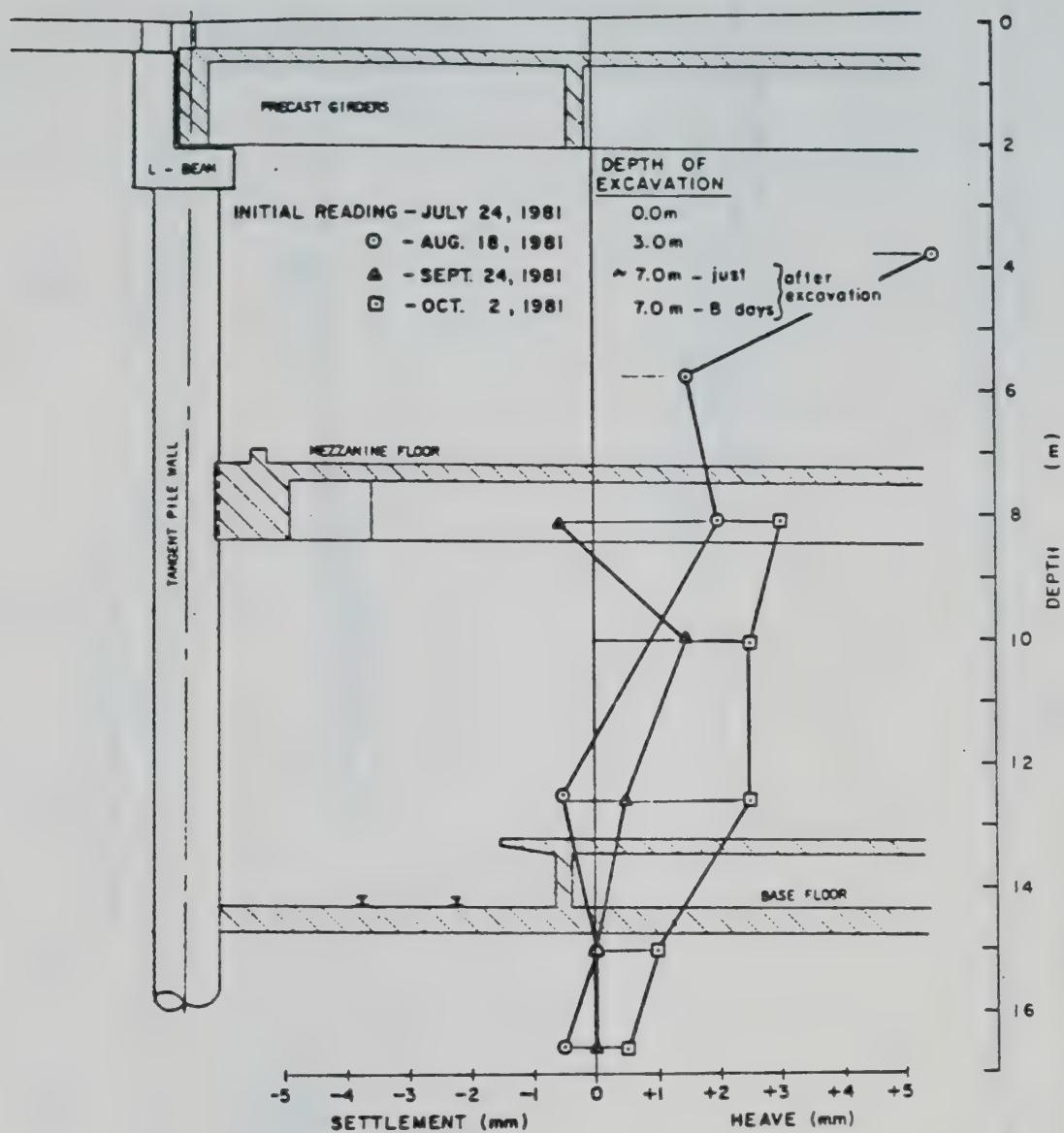
SOUTH LRT EXTENSION - CORONA STATION
SOIL HEAVE INSIDE THE STATION
MULTIPOINT EXTENSOMETER No. 1

Figure 4.4 Reading of multipoint extensometer in 107 Street Station (Thurber Consultants, 1980)



SOUTH LRT EXTENSION - CORONA STATION
SOIL HEAVE INSIDE THE STATION
MULTIPOINT EXTENSOMETER No. 2

Figure 4.5 Reading of multipoint extensometer in 107 Street Station (Thurber Consultants, 1980)



SOUTH LRT EXTENSION — CORONA STATION
SOIL HEAVE INSIDE THE STATION
MULTIPOINT EXTENSOMETER No. 3

Figure 4.6 Reading of multipoint extensometer in 107 Street Station (Thurber Consultants, 1980)

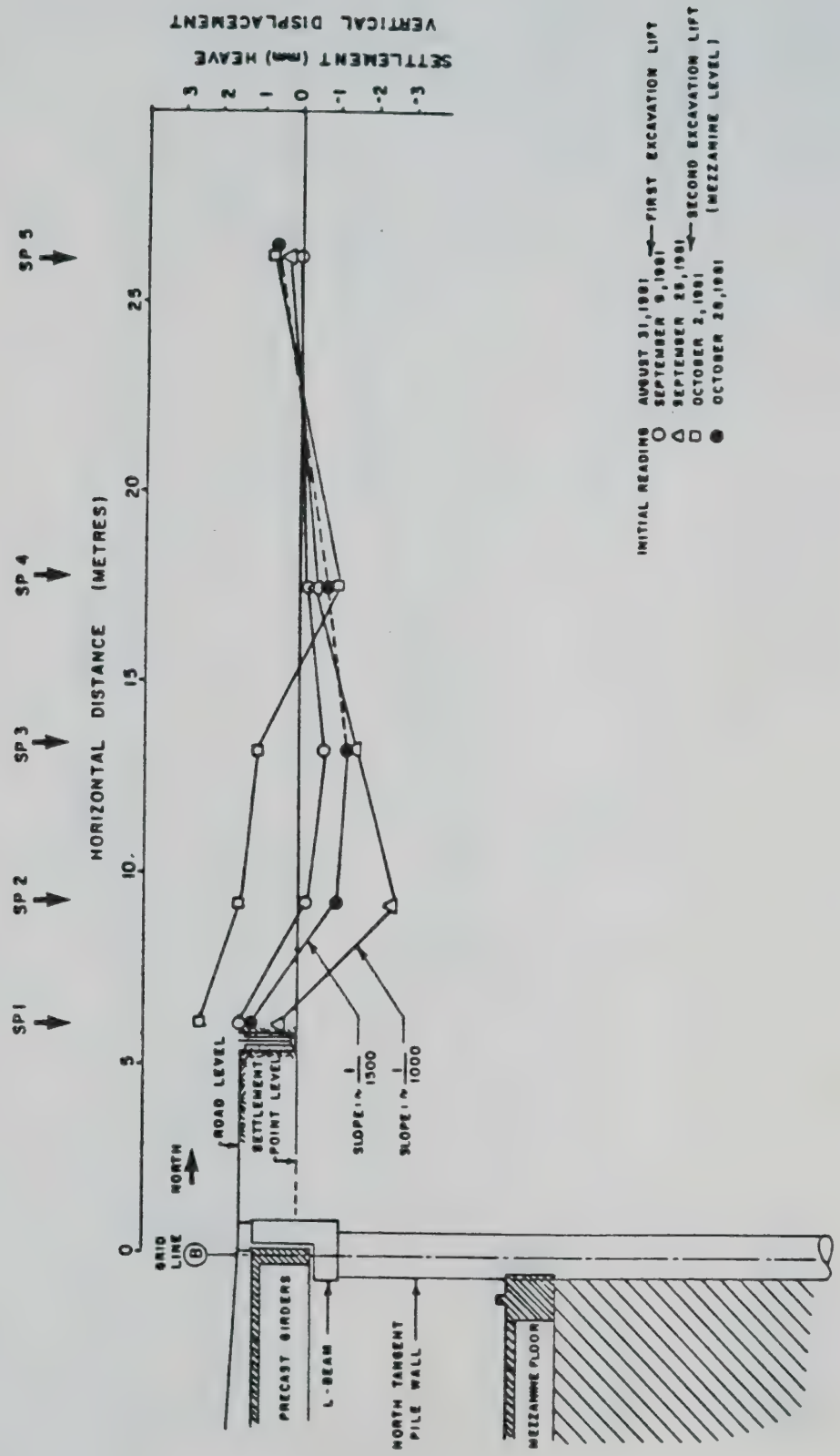


Figure 4.7 Measurement of settlement points in 107 Street
Station (Thurber Consultants, 1980)

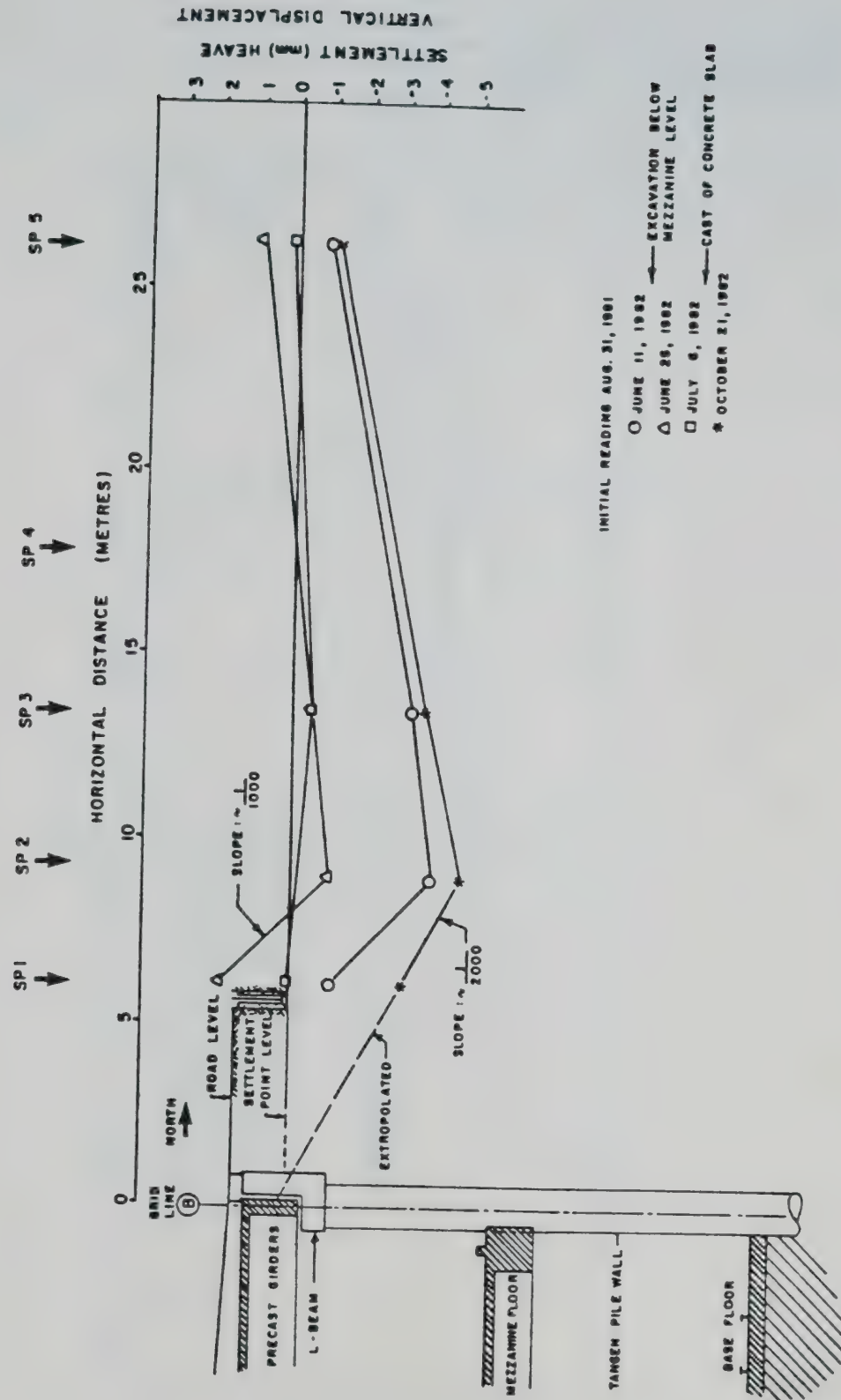


Figure 4.8 Measurement of settlement points in 107 Street Station (Thurber Consultants, 1980)

LEGEND

	DEPTH OF EXCAVATION BELOW G.S. (m)	DATE
—	0	11/8/81
—●—	8.4	23/4/82
—●—	14.7	16/6/82 ← EXCAVATION BELOW MEZZANINE LEVEL
—+—	14.7	13/10/82 ← CASTING OF CONCRETE BASE SLAB
—x—	14.7	25/7/83 POST CONSTRUCTION

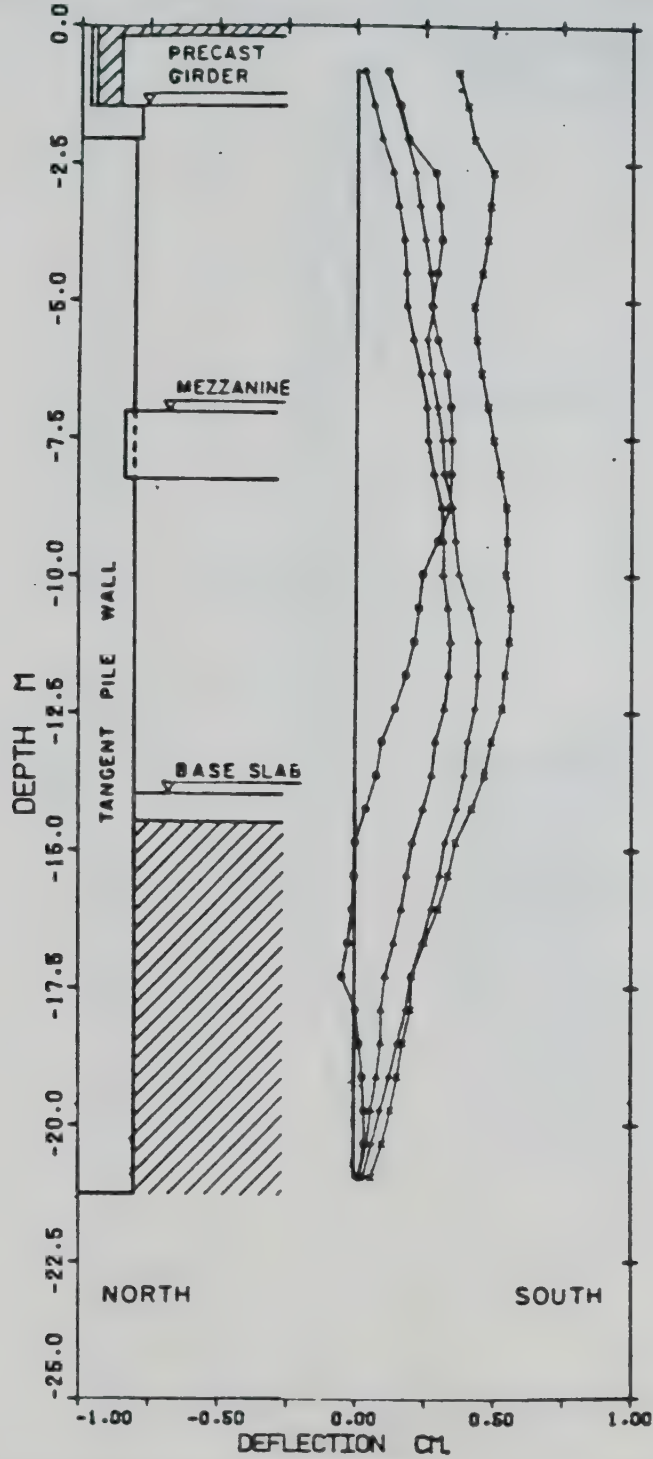


Figure 4.9 107 Street Station SI#1 (Thurber Consultants, 1980)

LEGEND

DEPTH OF EXCAVATION
BELOW G.S. (m)

DATE



0
8.4
14.7
14.7
14.7

11/8/81
23/4/82
16/6/82
13/10/82
25/7/83

POST CONSTRUCTION

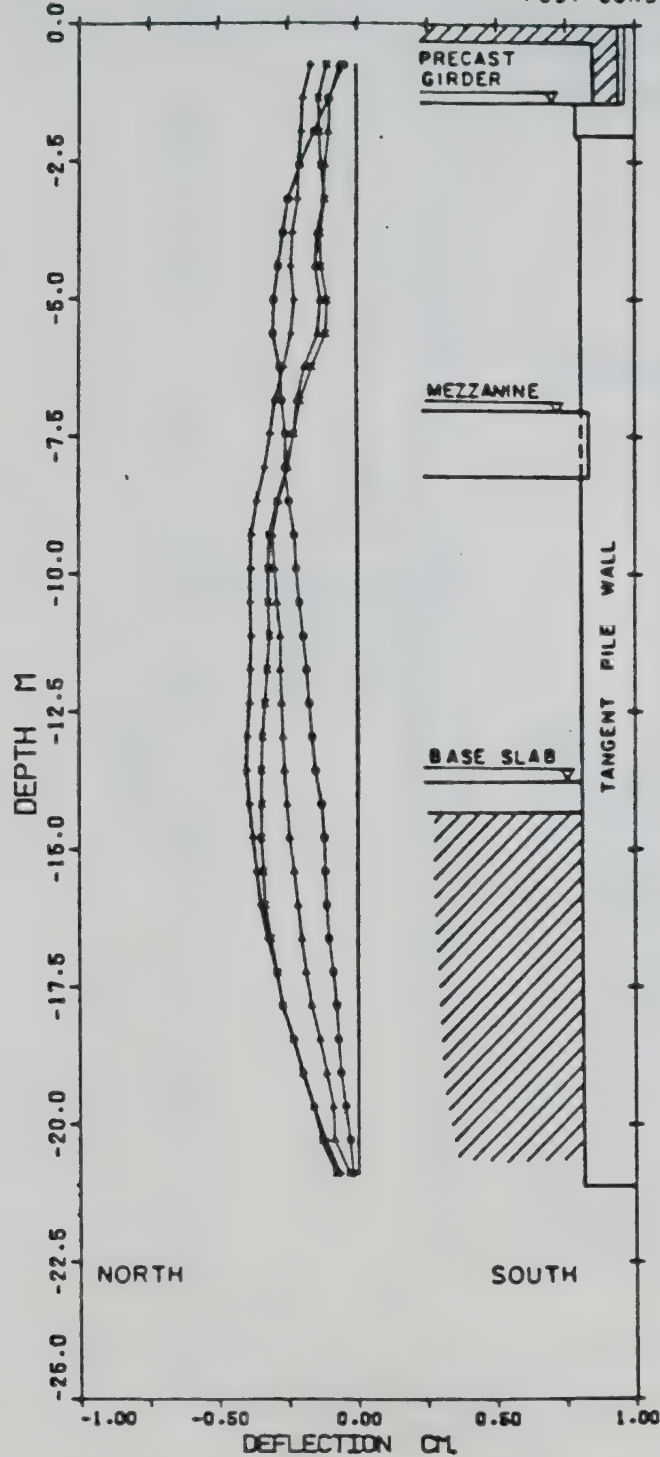


Figure 4.10 107 Street Station SI#2 (Thurber Consultants, 1980)

LEGEND

	DEPTH OF EXCAVATION BELOW G.S. (m)	DATE	
—	4.0	8/9/81	
—●—	7.5	9/5/82	
—●—	13.2	16/6/82	← EXCAVATION BELOW MEZZANINE LEVEL
—●—	13.2	13/10/82	← CASTING OF CONCRETE BASE SLAB
—X—	13.2	25/7/83	POST CONSTRUCTION

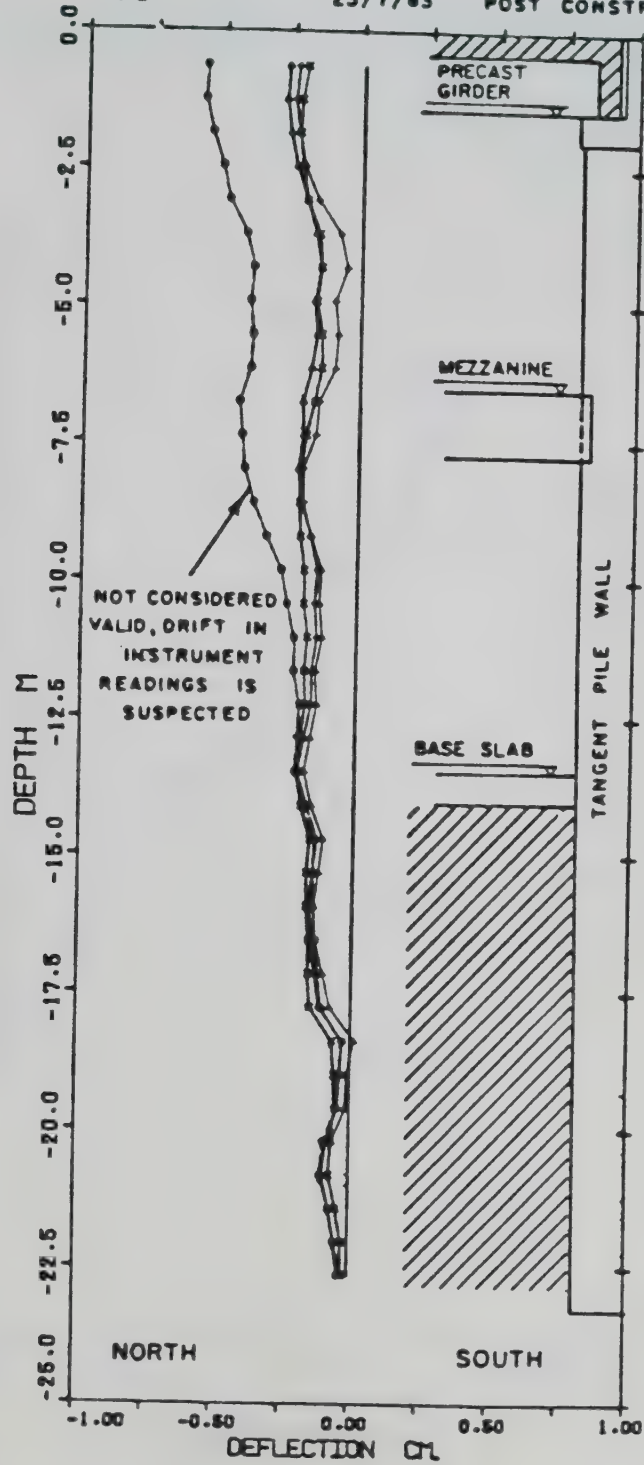


Figure 4.11 107 Street Station SI#3 (Thurber Consultants, 1980)

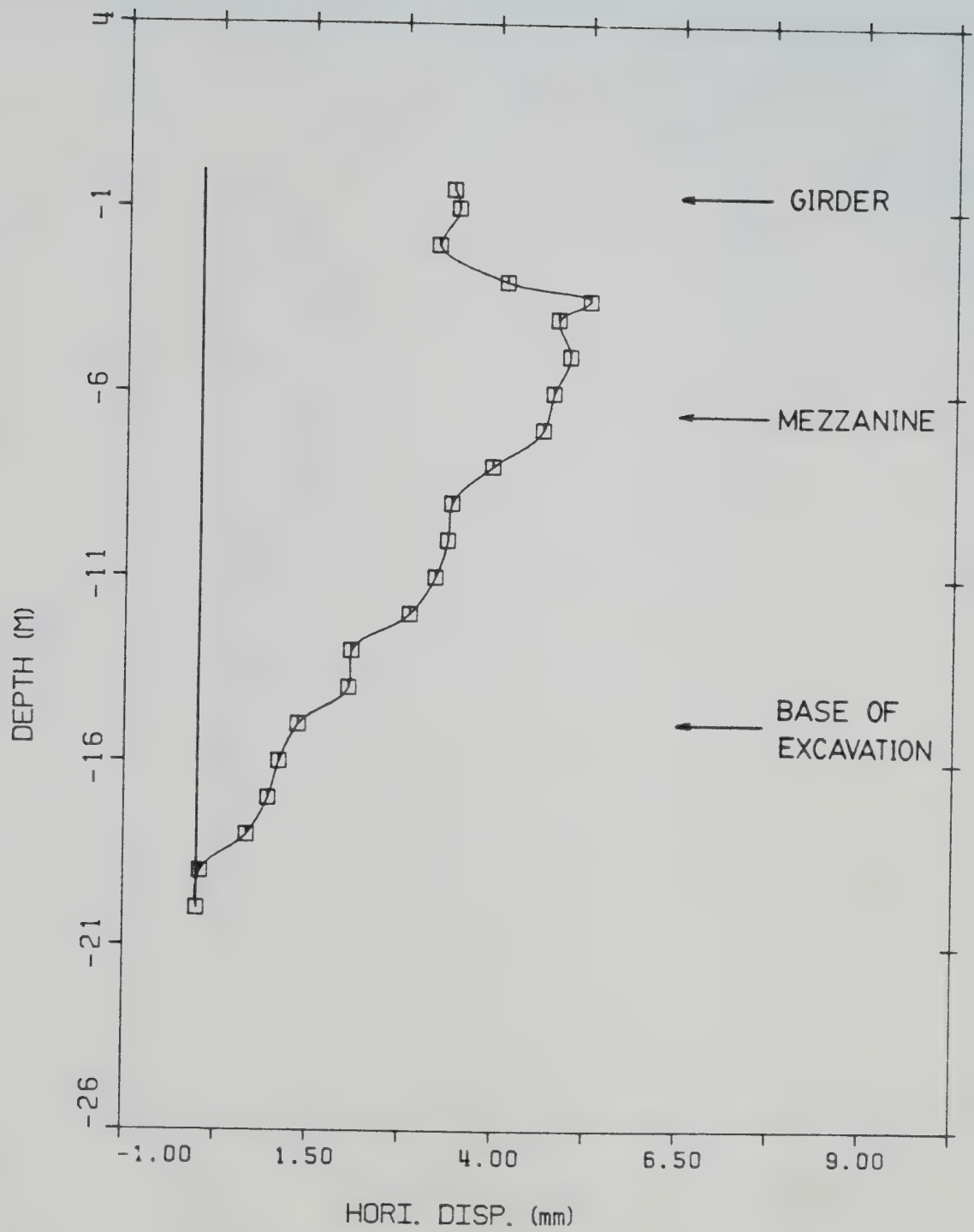


Figure 4.12 104 Street Station SI#088

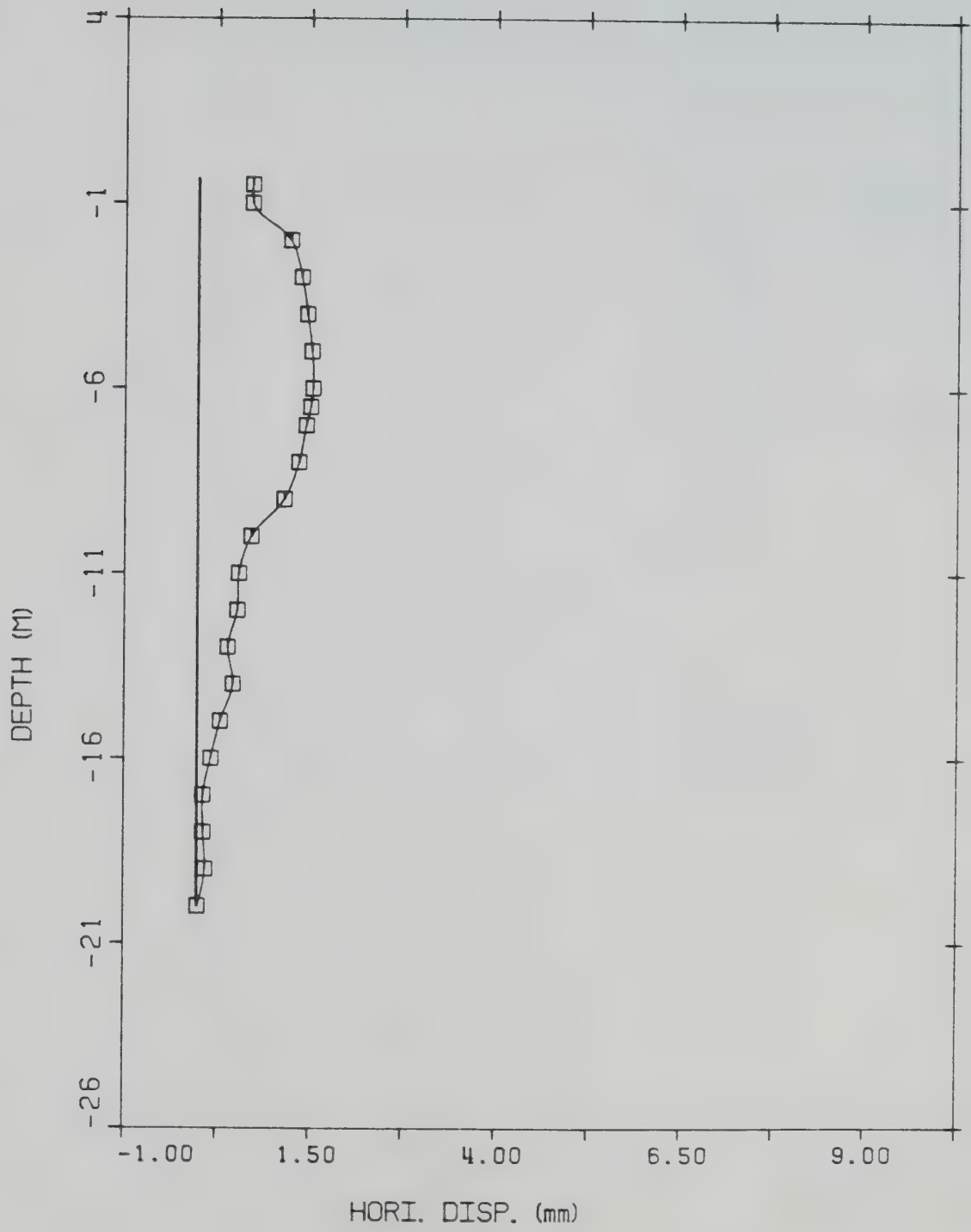


Figure 4.13 104 Street Station SI#103

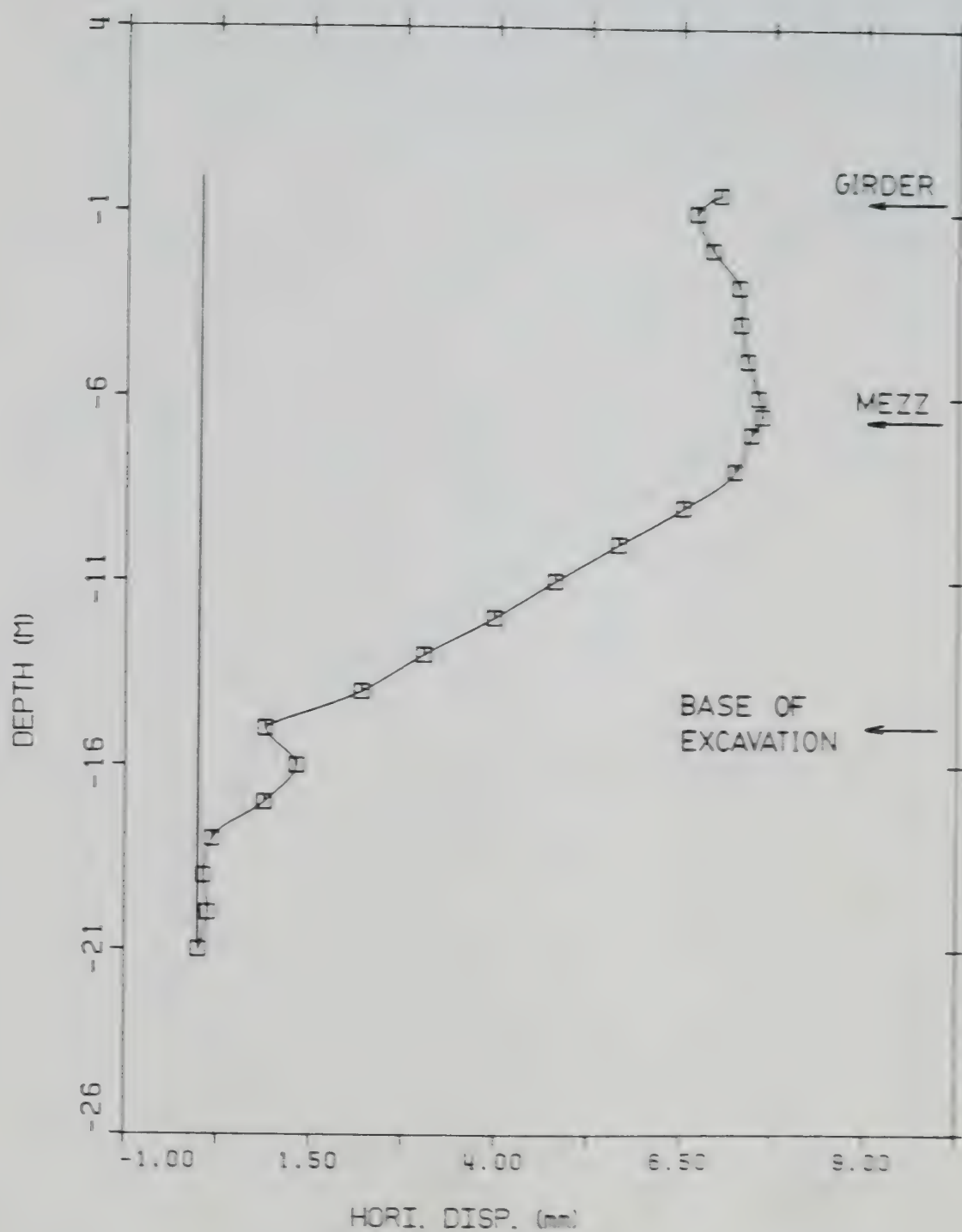


Figure 4.14 104 Street Station SI#203

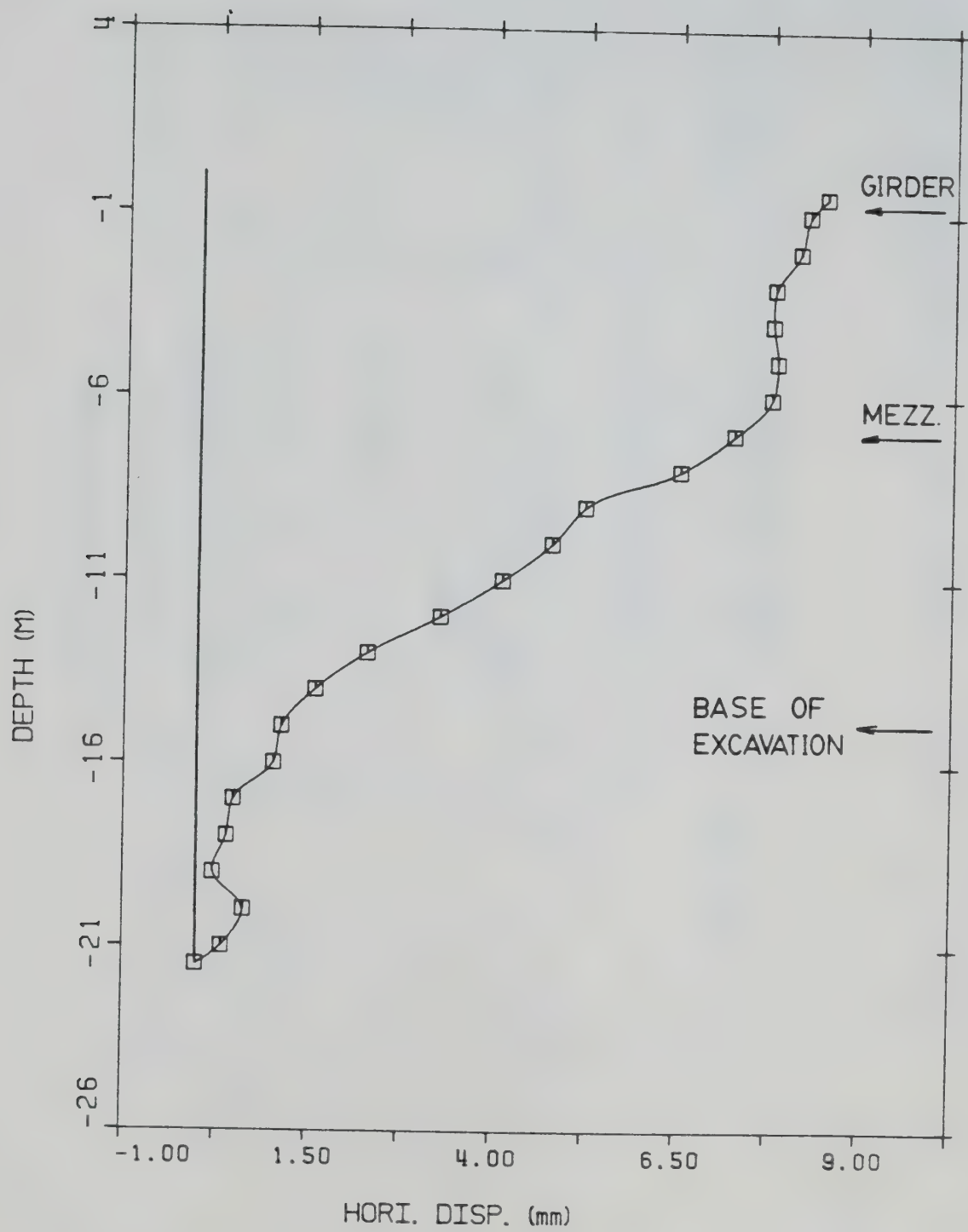


Figure 4.15 104 Street Station SI#243

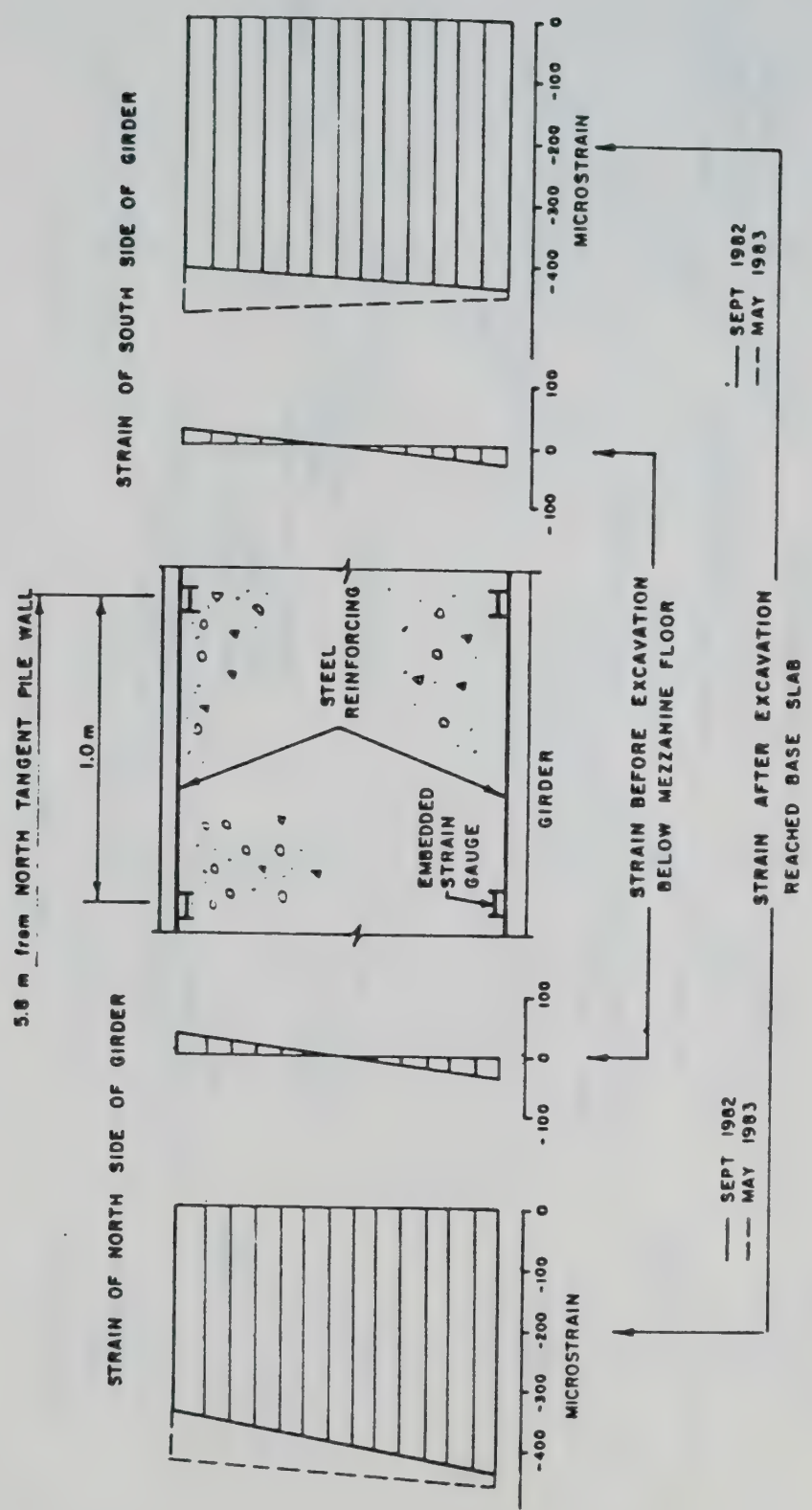


Figure 4.16 Measured strains in 107 Street Station (Thurber Consultants, 1980)

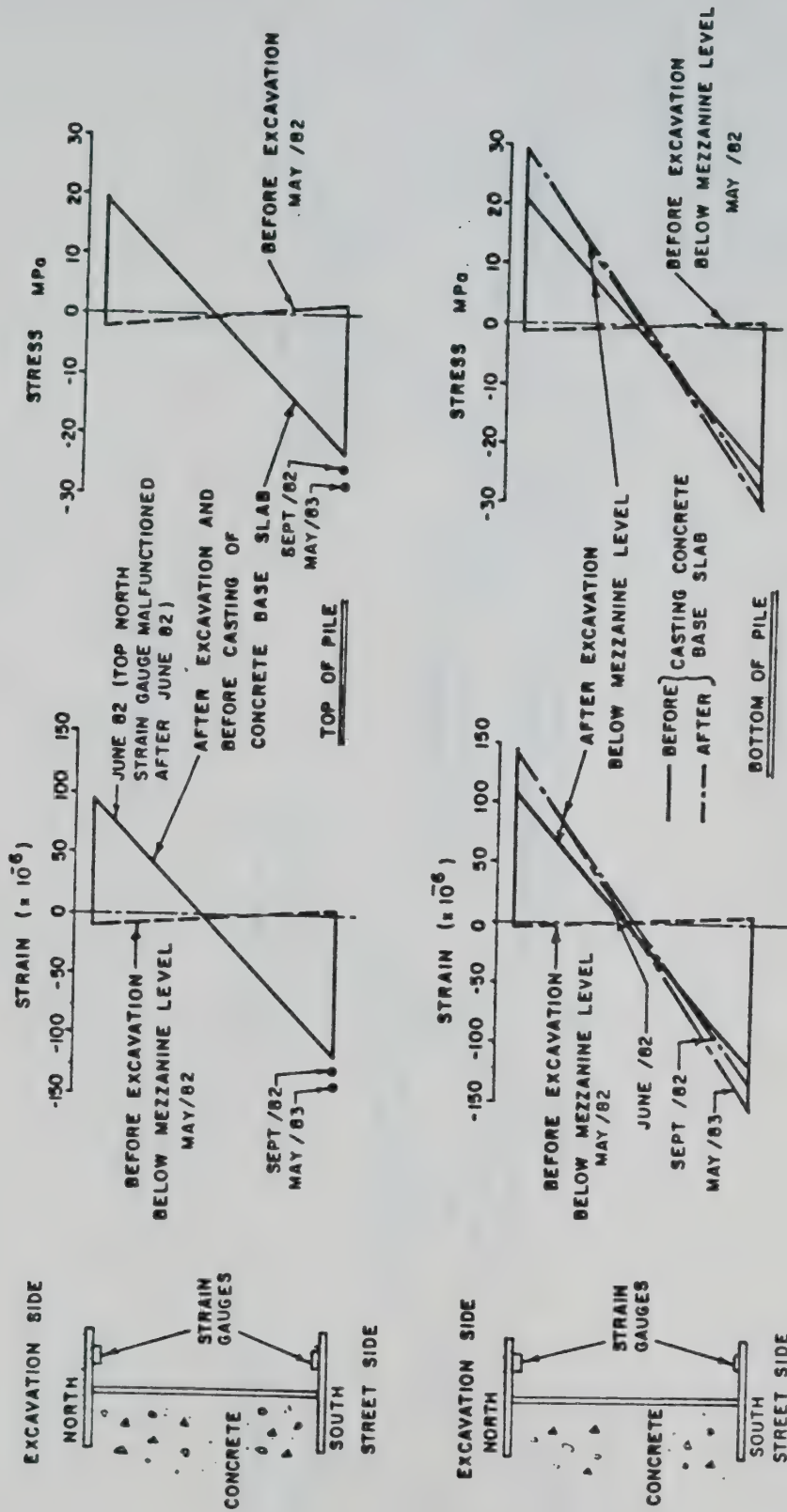


Figure 4.17 Weldable strain gauges (Thurber Consultants,

1980)

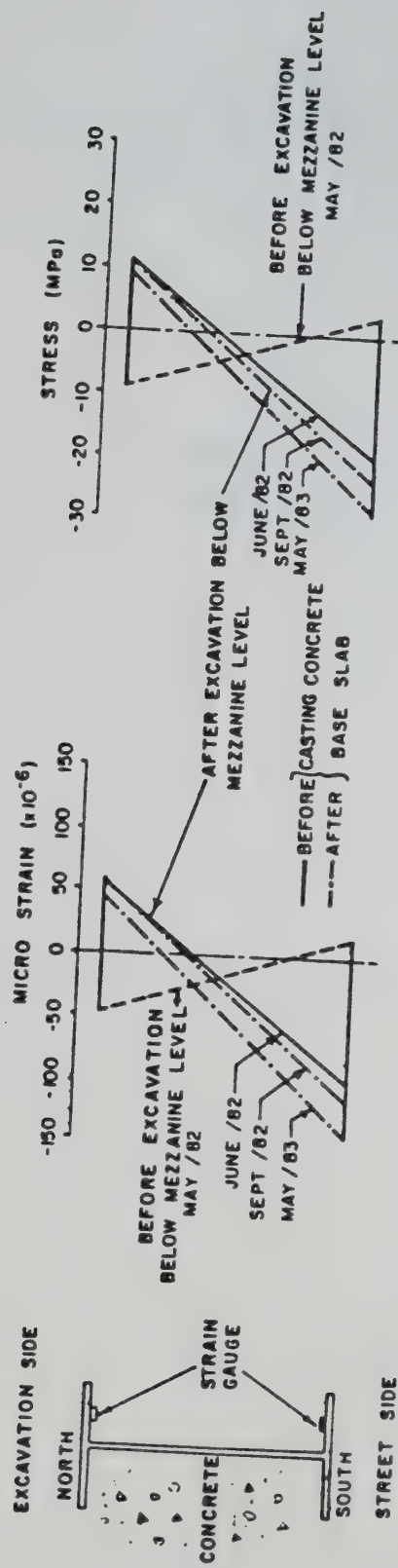


Figure 4.18 Weldable strain gauges (Thurber Consultants, 1980)

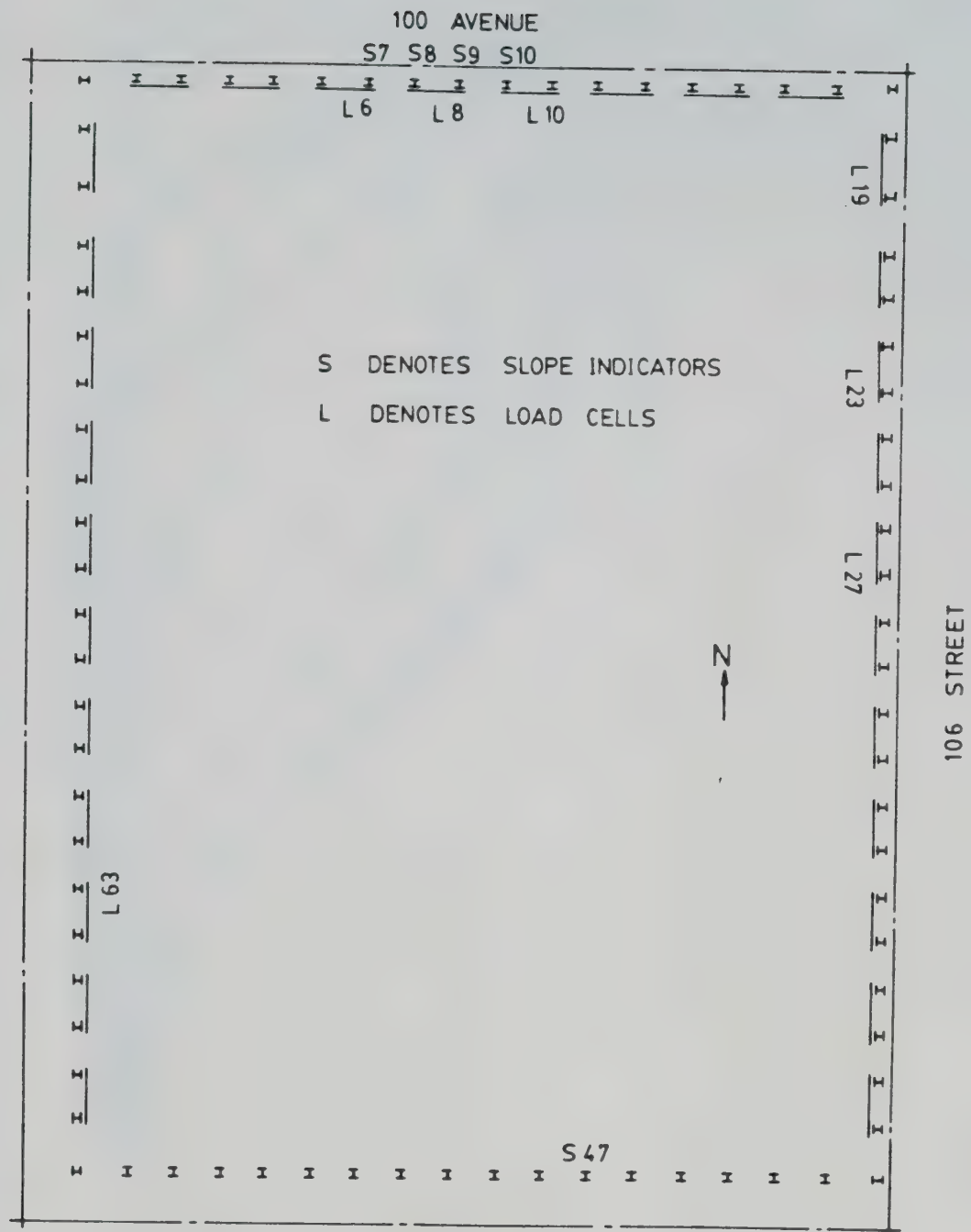


Figure 4.19 Layout of the instrumentation of the 106 Street and 100 Avenue (Sego, 1982)

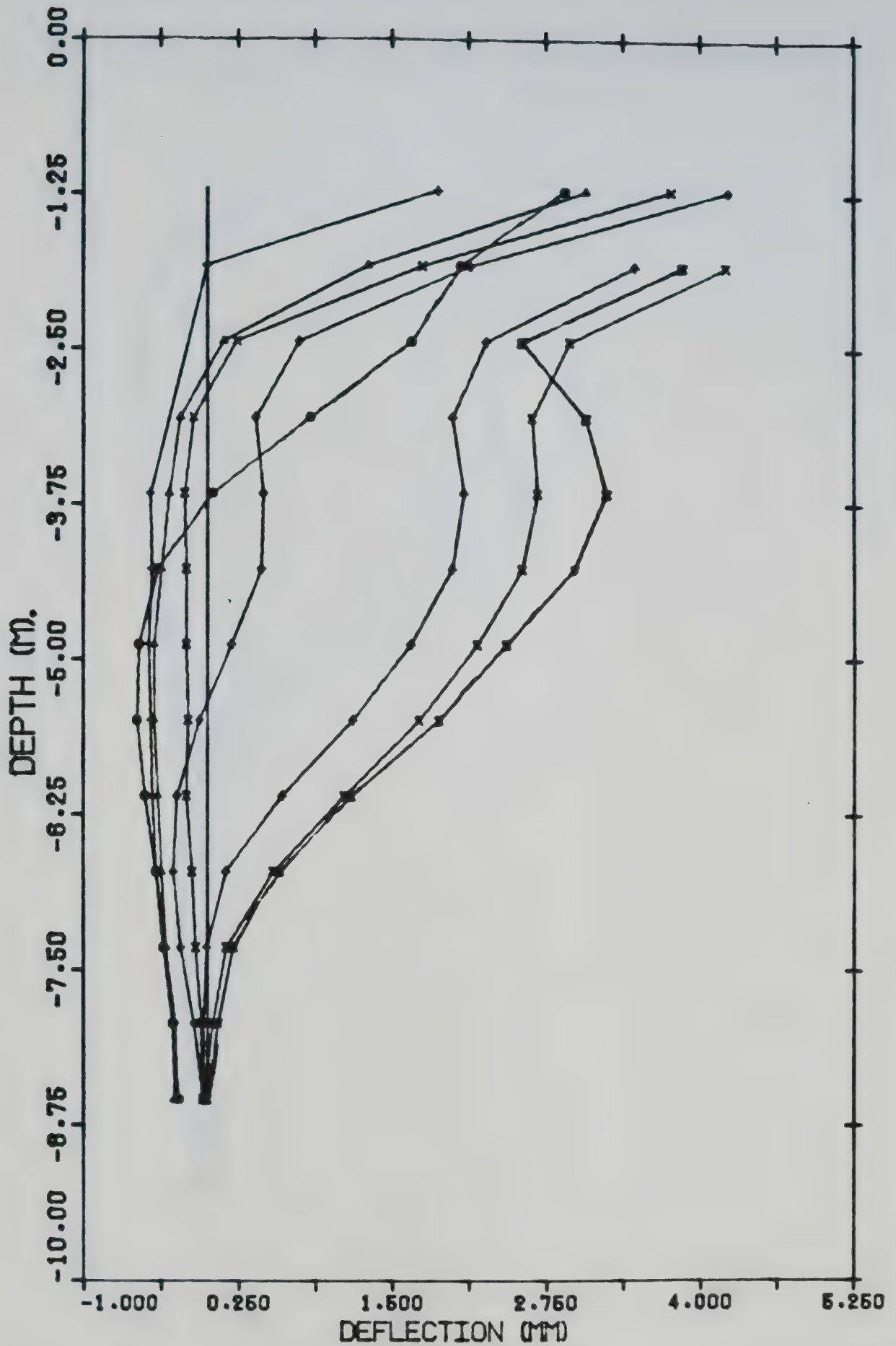


Figure 4.20 106 Street and 100 Avenue SI#7 (Sego, 1982)

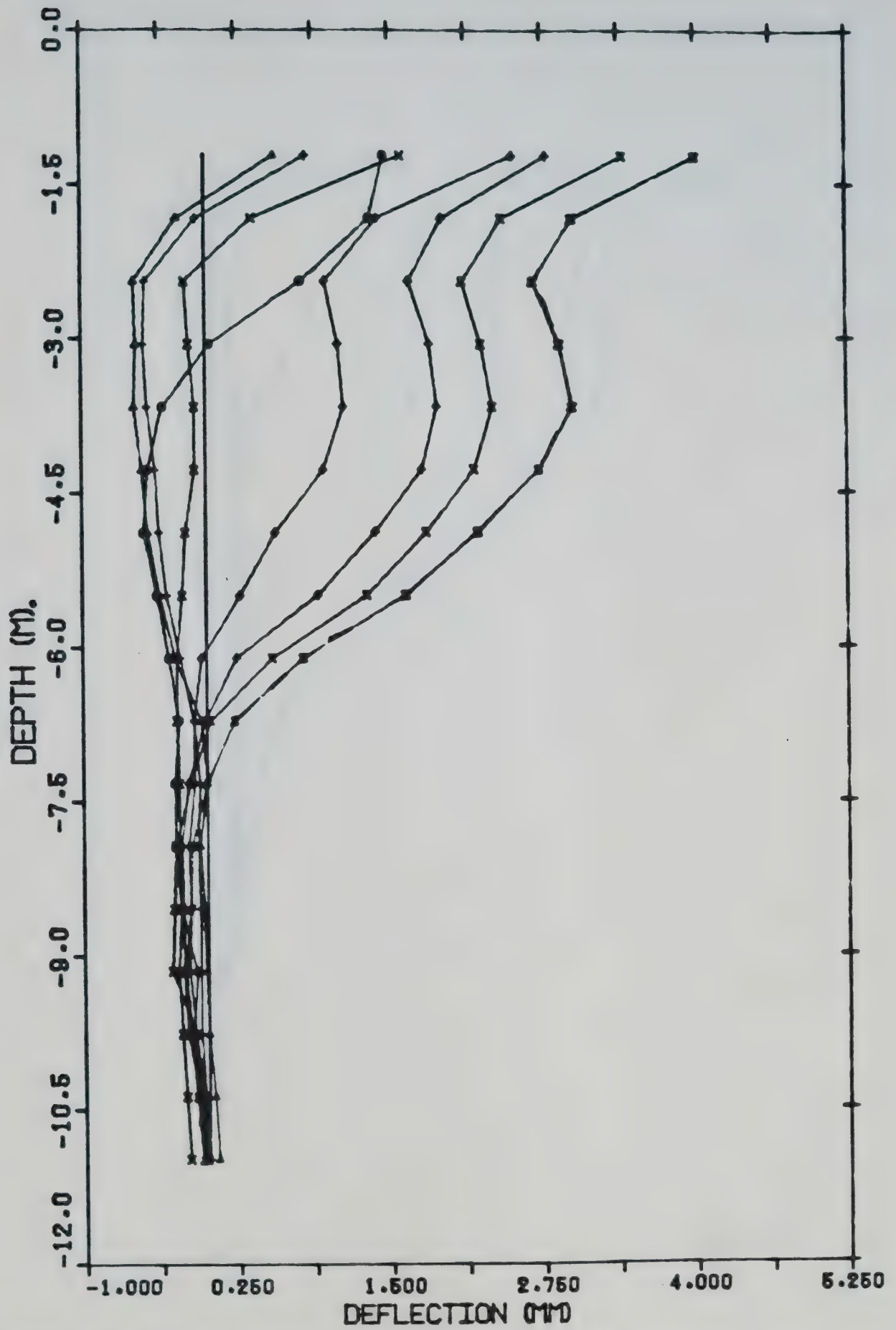


Figure 4.21 106 Street and 100 Avenue SI#8 (Sego, 1982)

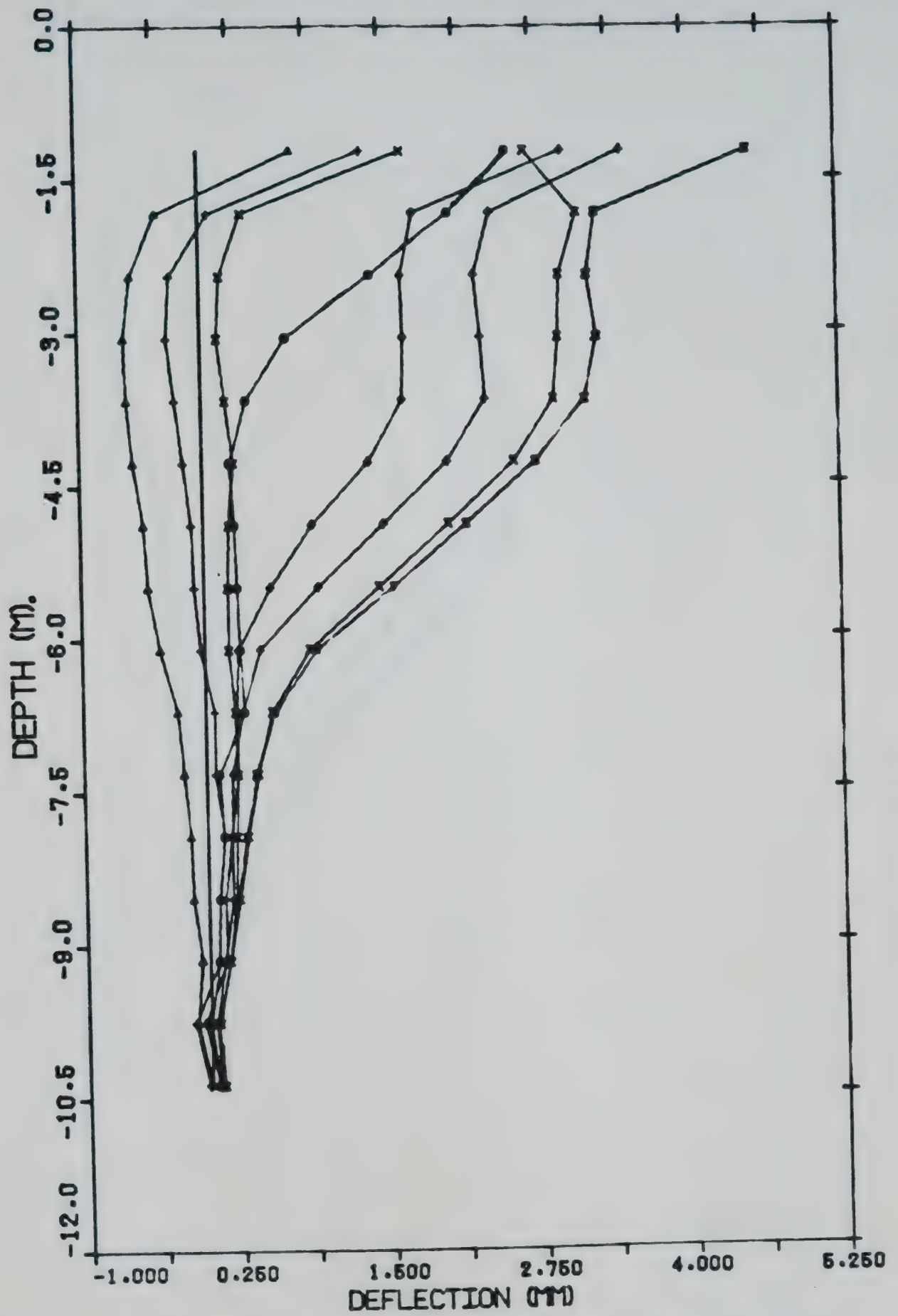


Figure 4.22 106 Street and 100 Avenue SI#9 (Sego, 1982)

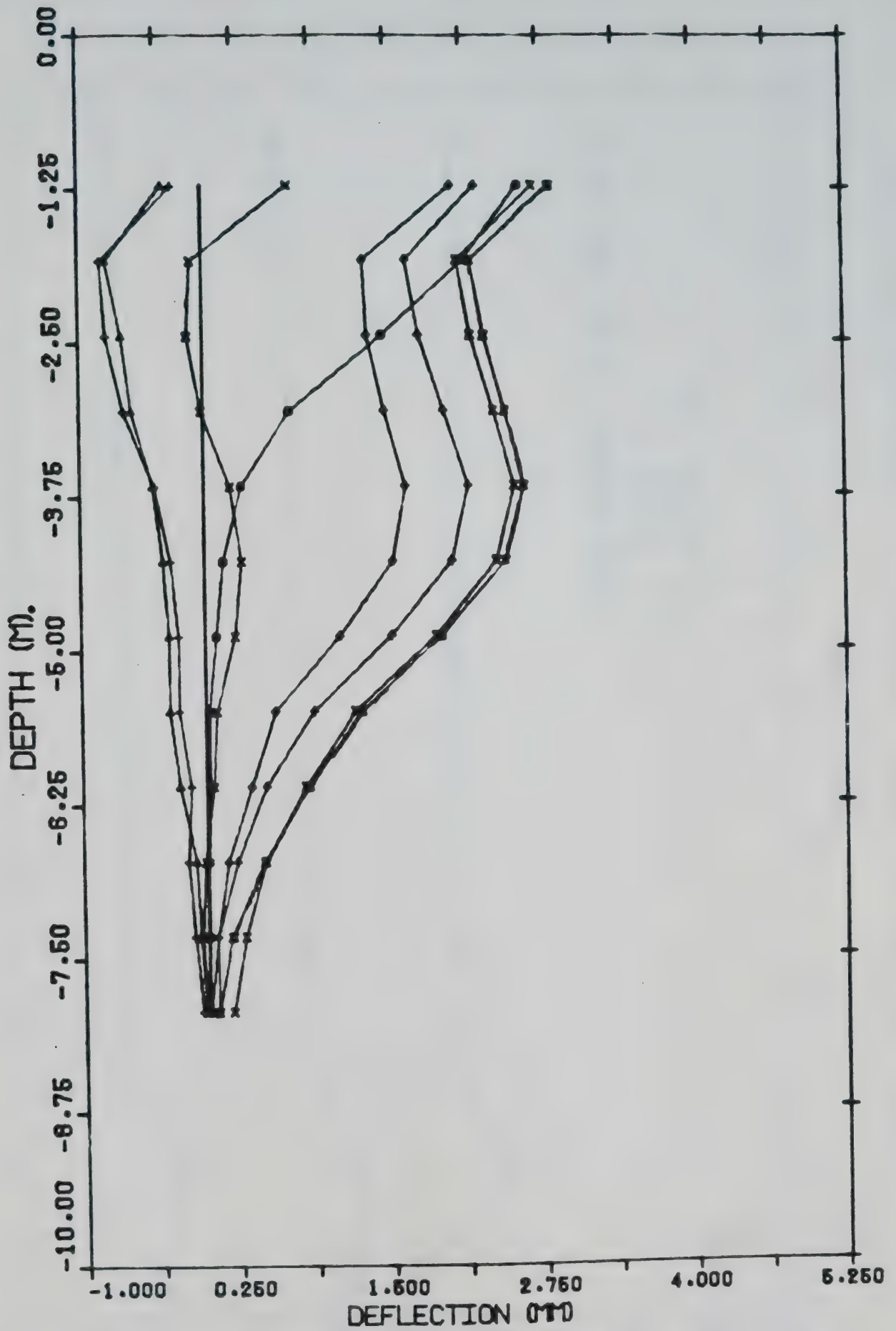


Figure 4.23 106 Street and 100 Avenue SI#10 (Sego, 1982)

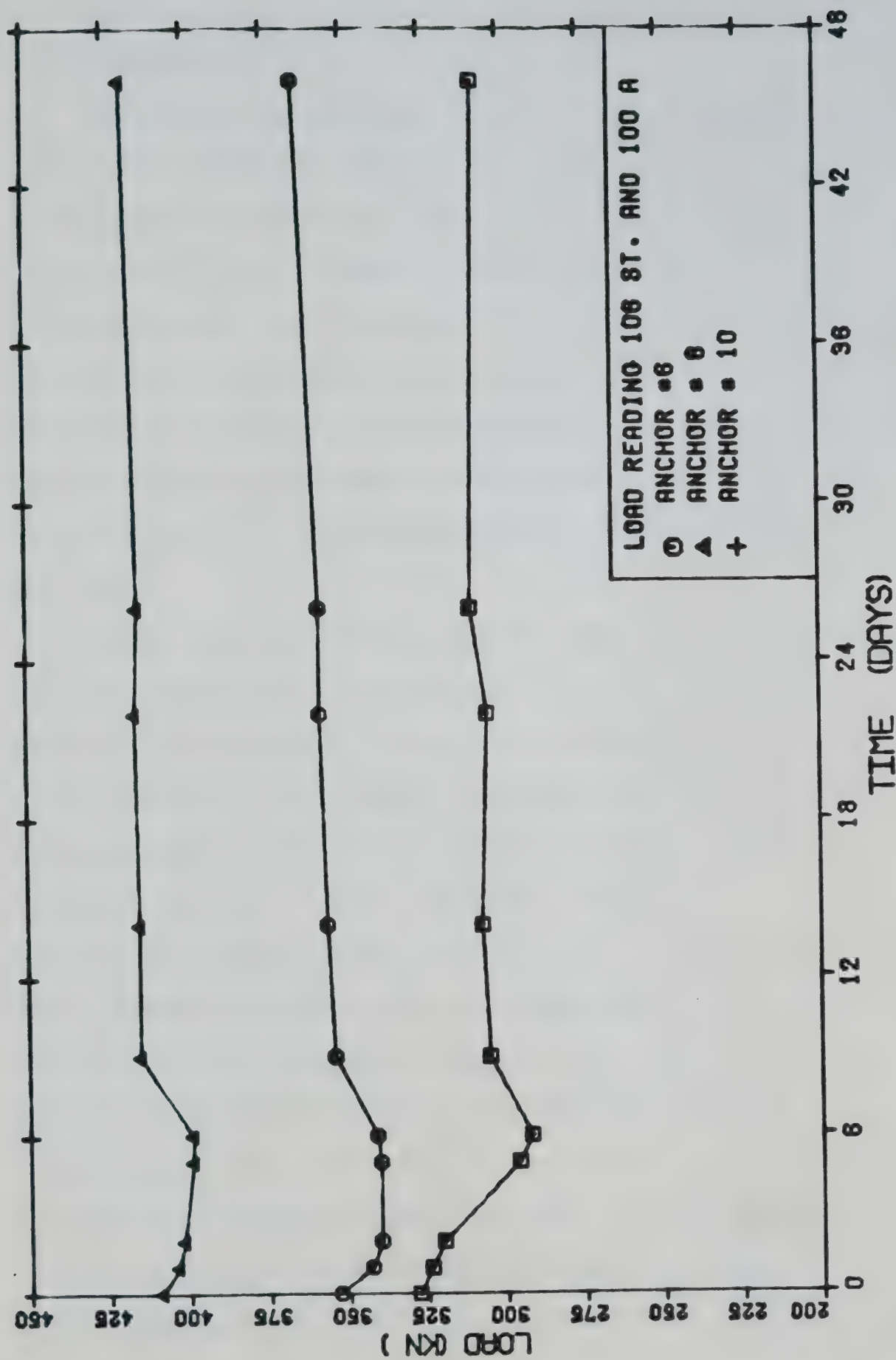


Figure 4.24 Load measurements of the H pile excavation (Sego, 1982)

5.0 Laboratory test data

5.1 Introduction

Before the development of electronic computers, the assumption of linear elastic soil behaviour was widely used to analysis stresses in a soil mass. This assumption obscured the significance of the stress path analysis in soils (Burland, 1977; Wroth, 1971). Due to the availability of high speed computer and powerful numerical analytical techniques, such as the finite element method, non linear analysis can be performed to represent the actual soil response prior to the development of shear planes within the soil mass.

Since the stress distribution behind a retaining wall was not well known, the concept of safety against a total failure has previously been used in design. Later, empirical earth pressure distribution were proposed based on field measurements, such as the apparent pressure diagram of Terzaghi and Peck (1967) and Peck (1969). By introducing the concept of safety against failure into the design, only the shear strength of the soil was required. Design base on empirical earth pressure diagram only required the unit weight of the soil to obtain the design pressure distribution. The variation in the stress strain behaviour of soil with water content, density, drainage condition, structure, strain conditions, duration of loading (consolidation), stress history, confining pressure and

shear stress were not considered in the design.

Analytical studies using finite element method as described in chapter 2 showed that the concept of stress path was aided in describing most aspect of soil behaviour (Eisenstein and Medeiros, 1983). Figure 5.1 shows the basic idea of how an excavation was performed in the finite element analysis. The excavation was represented by removing the boundary forces equal and opposite to the initial state of stress along the excavation surface. The material was assumed elastic. After each removal of forces, a change of stress and strain occurs in the elements. The elastic constants change according to the stress strain relationship during the nonlinear analysis. The same approach was repeated until the final depth of excavation was reached.

In nonlinear analysis, the three distinct stress path as shown in figure 5.2 can be established around the excavation. The horizontal stress of the soil besides the vertical wall decreases due to the removal of soil while the vertical stress exhibit no significant change. At the bottom of the excavation, the condition is reversed. A decrease in vertical stress with the horizontal stress almost unchange was observed. Zone C as indicated in the diagram representing condition between the two described above. The soil in this zone decreases both in the horizontal and vertical in the same proportion in the early stage and later with vertical stress constant while the horizontal stress decreases.

Medeiros (1979) set up a laboratory experimental program to run various tests to reproduce the actual field behaviour of soil around a tangent pile wall. The tests were active compression (triaxial and plane strain), active extension and proportional unloading as described by figure 5.3. Conventional triaxial tests, with increasing vertical stress and constant horizontal stress, were made for comparison purpose only. In actual field conditions, this stress path was not followed although it was the easiest test to perform.

These tests discussed above were made to investigate the difference between the stress strain relationship for different stress path. For the same stress level, Green (1971) showed that the modulus of deformation in a plane strain test was higher than in convention triaxial test.

The data obtained by Medeiros (1979) will be used in analysing the case histories in this study and is described in the following sections in this Chapter.

5.2 Till

The till was typically composed of 25% clay, 34% silt and 41% sand. The average liquid limit and plastic limit were 35% and 15.5% respectively. The specific gravity and water content were found to be 2.69 and 14%.

A coefficient of earth pressure at rest of 0.85 was used for the experiments. The samples were tested under the following stress paths and the data are:

1. Passive compression

The samples were first subjected to an isotropic confining pressure 10% higher than the overburden pressure and then the vertical stress was gradually increased without changing the lateral stresses. The laboratory results were shown in figure 5.4 to 5.6. The angle of shearing resistance obtained was 40° .

Wittebolle (1983) presents results of tests performed on samples from the building site in 100 Street and 106 Avenue. Figure 5.7 to 5.9 contains the test data. The angle of shearing resistance obtained was 37° .

2. Active compression

The samples were initially consolidated isotropically. The confining pressure was then reduced with a increase in the axial stress to compensate for the relief in the cell pressure. Figure 5.10 to 5.12 show the results of the tests. The angle of shearing resistance was not presented.

3. Plane strain passive compression

The tests were performed under the same procedures as the triaxial passive compression test in the specially designed plane strain apparatus (Medeiros, 1979). Figure 5.13 to 5.15 contain the stress strain curves with an average angle of shearing resistance of 46.5° .

4. Plane strain active compression

The samples were consolidated anisotropically with a ratio 0.85. The tests were performed by using the plane strain apparatus with results shown in figure 5.16 and 5.17. The shearing resistance was not calculated.

The angle of shearing resistance obtained from passive compression in triaxial and in plane strain showed a difference in 6.5° which represent a significant difference in shear strength.

5.3 Relationship between triaxial and plane strain passive compression

For linear elastic soil, the stress strain relationship could be described by the equation:

$$\Delta\epsilon_1 = [\Delta\sigma_1 - \mu(\Delta\sigma_2 + \Delta\sigma_3)] / E \quad (5.1)$$

where

ϵ_1 = major principal strain

$\sigma_1, \sigma_2, \sigma_3$ = principal stresses

E = Young's modulus

μ = Poisson's ratio

Since the confining pressure in the passive compression test was constant, equation 5.1 can be rewritten as:

$$\Delta\epsilon_1 = \Delta\sigma_1 / E \quad (5.2)$$

For plane strain condition, equation 5.1 reduces to:

$$\Delta\epsilon_{1ps} = (1 - \mu^2) (\Delta\sigma_1 - \Delta\sigma_3 \mu / (1 - \mu)) / E \quad (5.3)$$

where

$$\Delta\epsilon_2 = 0$$

For passive compression in plane strain, equation 5.3 can be further reduced with constant confining pressure ($\Delta\sigma_3=0$):

$$\Delta\epsilon_{1ps} = \Delta\sigma_1 (1 - \mu^2) / E \quad (5.4)$$

or

$$\Delta\epsilon_{1ps} = \Delta\sigma_1 / E_{ps} \quad (5.5)$$

where

$$E_{ps} = E / (1 - \mu^2)$$

5.4 Summary of tests on till

The modulus of deformation represented by the tangent of the stress strain curves in triaxial and plane strain passive compression tests were then related by equation 5.5. A comparison between the modulus of deformation of the two tests is shown in figure 5.18. The initial ratio of the modulus was about 1.25 while the theoretical value for a poisson ratio of 0.42 would be 1.21. As it can be observed from the plot the ratio increases with increasing deviator

stress.

A similar comparison between triaxial and plane strain in active compression was impossible since the plane strain tests were consolidated anisotropically where as the triaxial test were not and therefore they were tested with different initial conditions. The modulus of deformation was compared with change in strains instead of stress changes. Figure 5.19 showed that a significant difference was observed between the passive and active compression tests.

5.5 Sand

The Saskatchewan sand is composed of 95% sand and 5% silt and clay. The average moisture content was 5% and the degree of saturation is 28%. The unit weight of the soil determined was 18.7 kN/m^3 and the specific gravity was 2.67.

1. Passive compression

Initially, the samples were consolidated isotropically to the field overburden pressure (figure 5.20). The hyperbola fitting were not satisfactory. The average angle of shearing resistance observed was 40.5° and the strain at failure was about 3%.

2. Active extension

The samples were consolidated anisotropically to the overburden pressure with a K_0 value of 0.36 which was equal to $1 - \sin\phi$. The angle of shearing resistance was the same as in passive compression while the failure strain was reduced to 0.9%. Figures 5.21 to 5.23 show

the results of the tests.

3. Proportional active compression

The sample was consolidated anisotropically and the principal stresses were then reduced at a constant ratio. Finally, a reduction in minor principal stress as in active compression tests was performed. Figure 5.24 shows the result of the test.

5.6 Summary of tests on sand

A comparison between the modulus of deformation between passive compression and active extension tests was difficult due to the difference in consolidation ratio. Therefore it was only compared after the samples in extension reached the isotropic state of stress. Figure 5.25 shows the modulus of the two tests with close values of overburden pressure and it also show a significant difference between two curves.

As observed in active extension tests, the strains to failure are smaller than in passive compression tests which indicates the shear strength mobilized earlier.

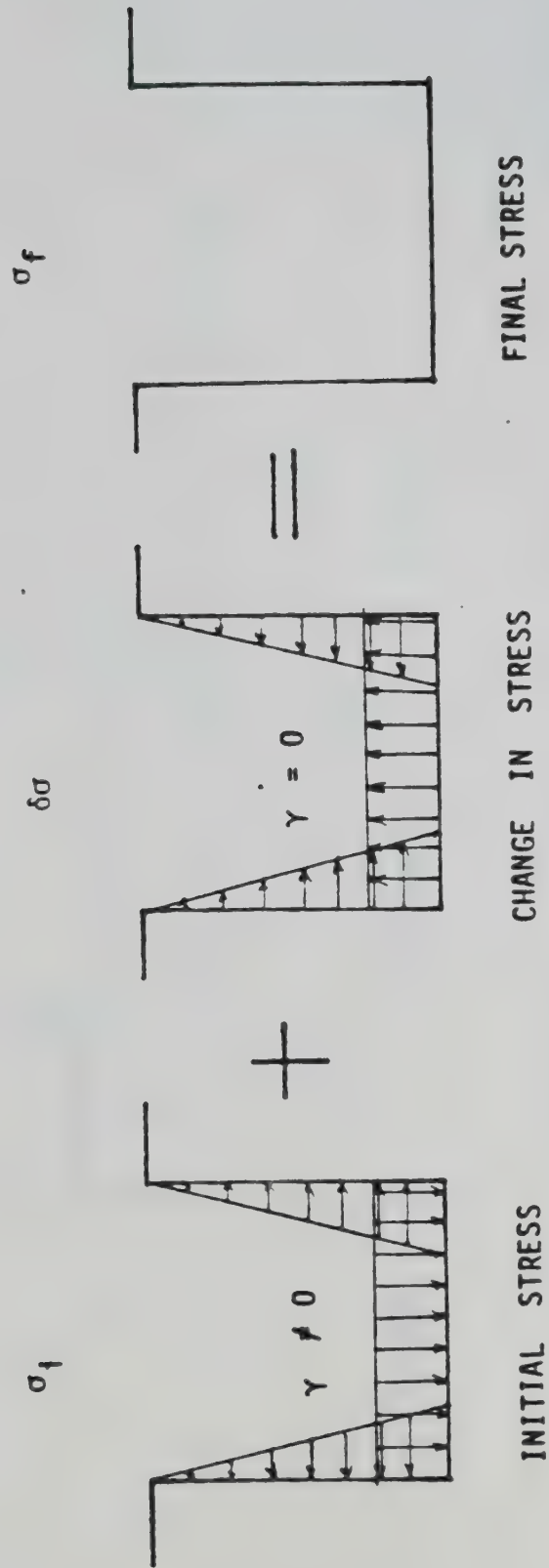


Figure 5.1 One step analysis of excavation (After Medeiros, 1979)

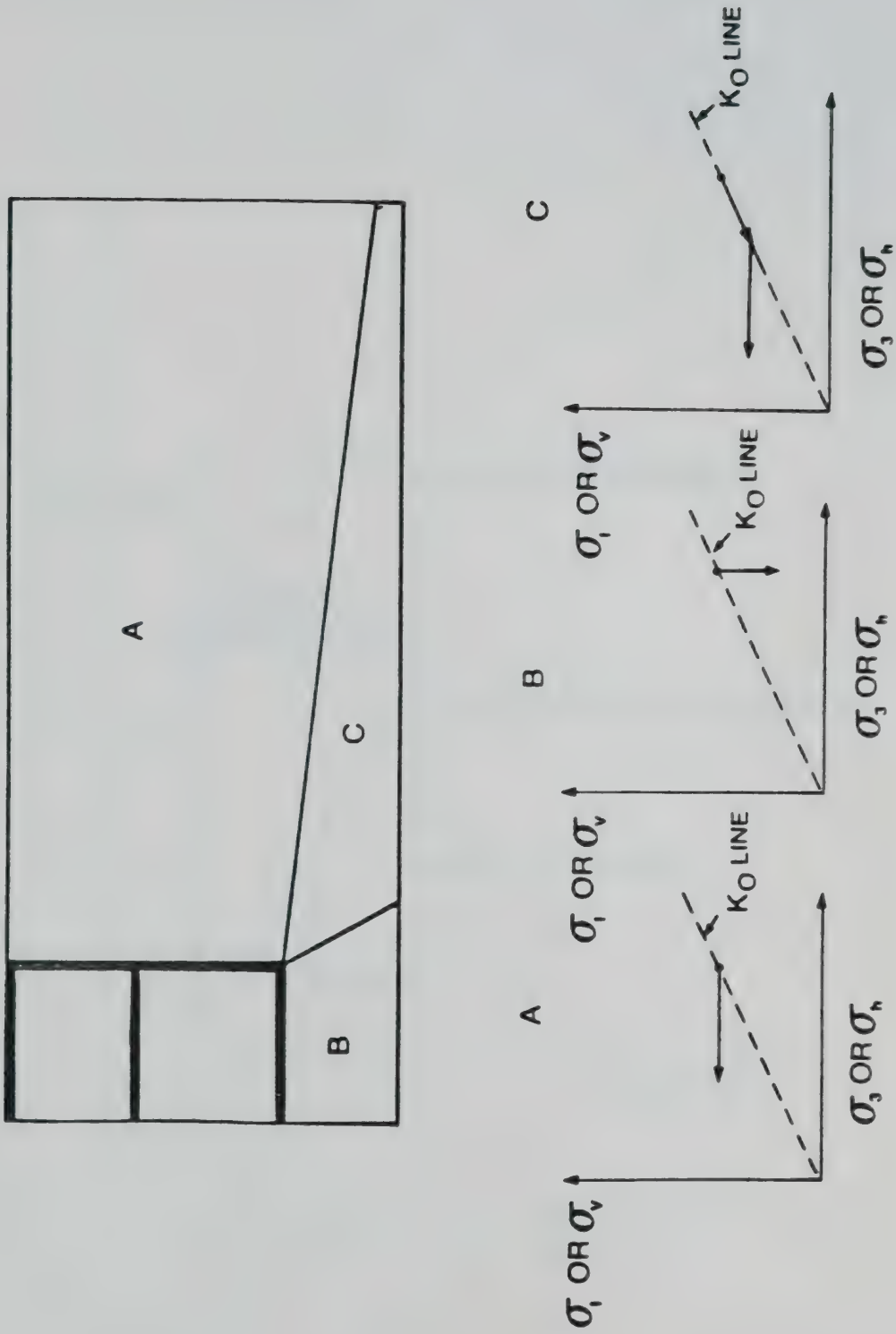


Figure 5.2 Stress path involved in excavation (After Medeiros, 1979)

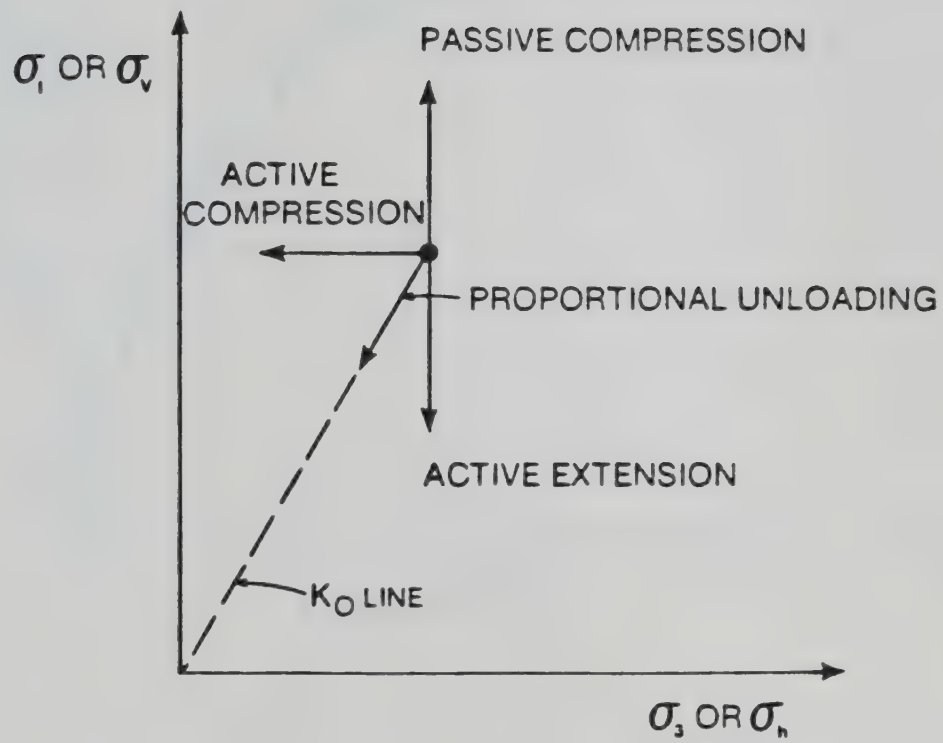
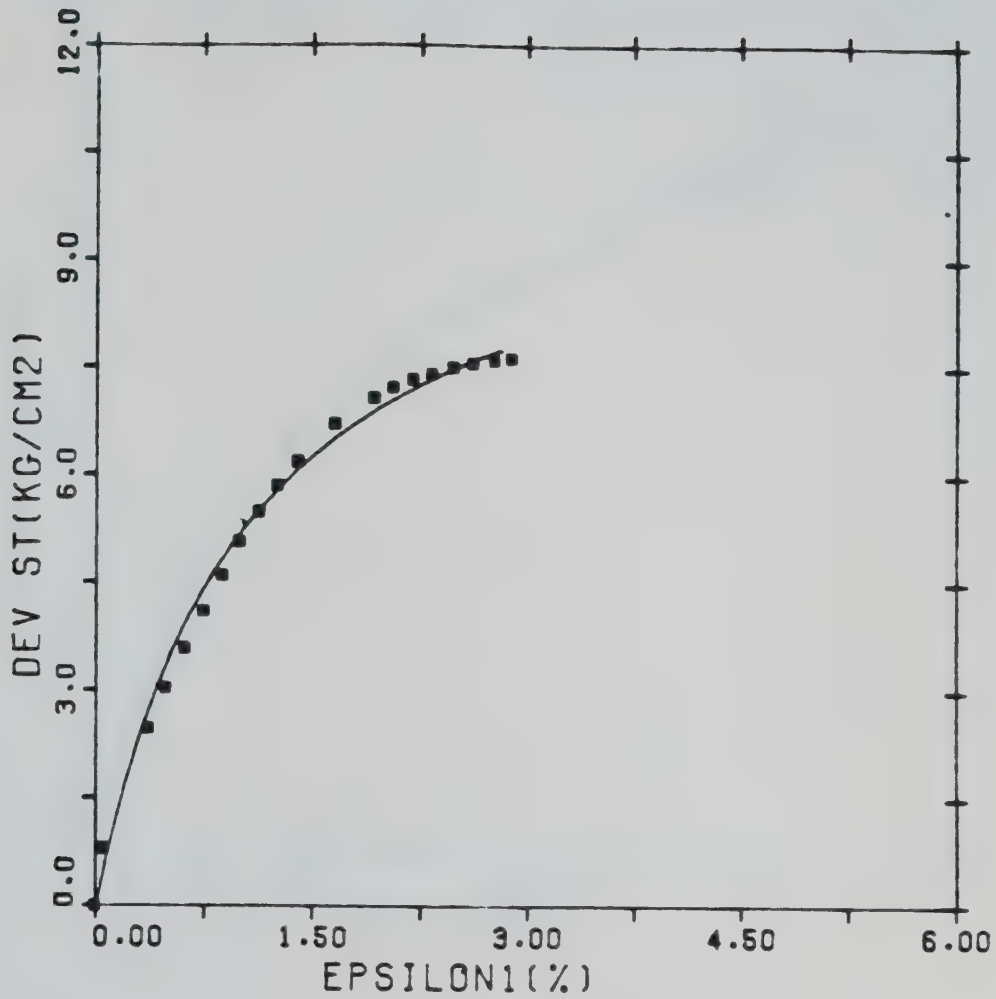
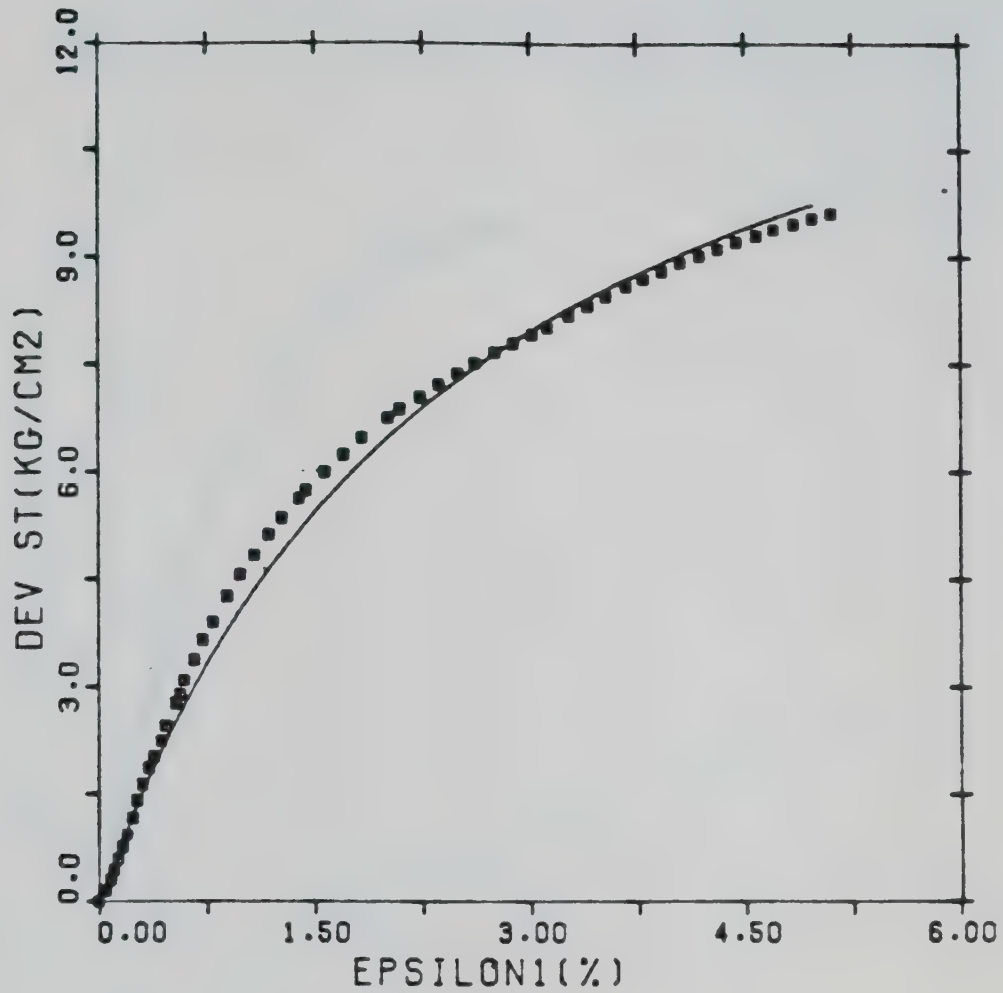


Figure 5.3 Stress path investigated (After Medeiros, 1979)



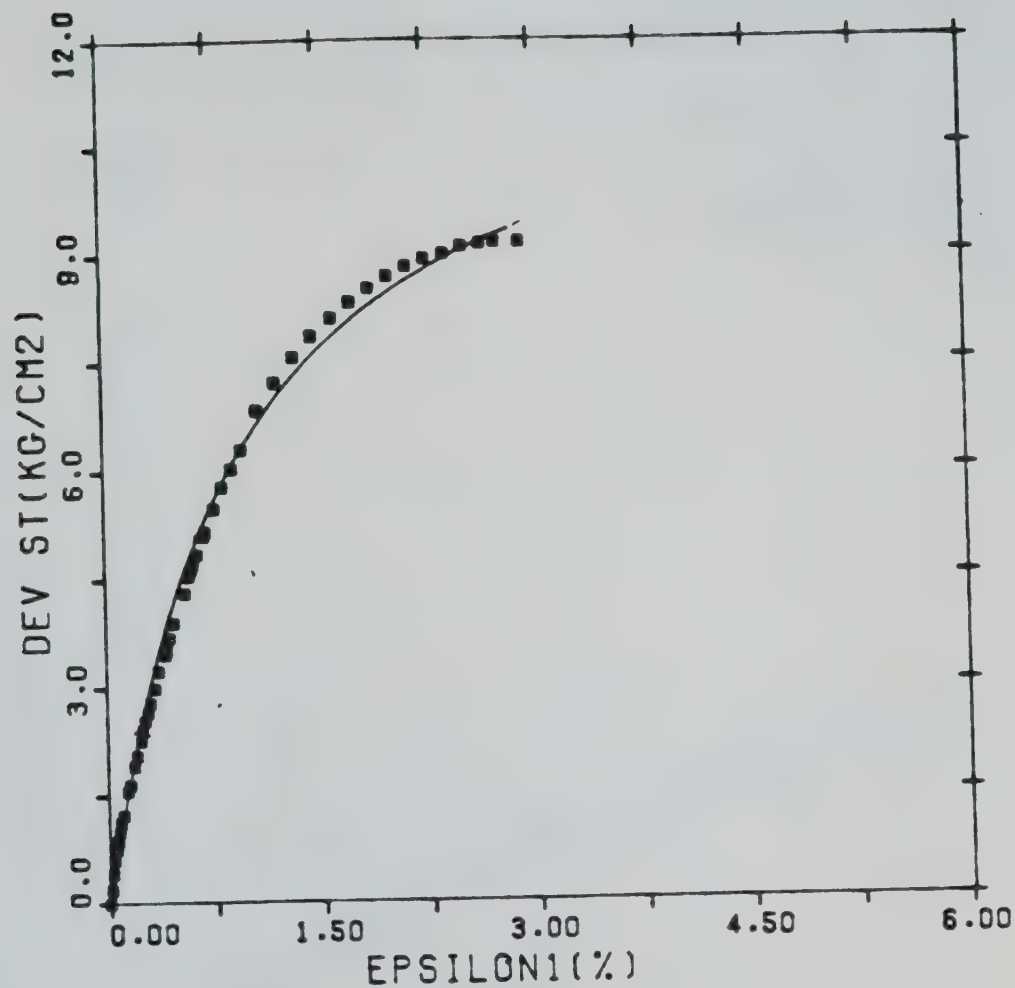
COMPRESSION SIGMA3=1.540

Figure 5.4 Passive compression test in till (After Medeiros, 1979)



COMPRESSION SIGMA3=2.765

Figure 5.5 Passive compression test in till (After Medeiros, 1979)



COMPRESSION SIGMA3=3.360

Figure 5.6 Passive compression test in till (After Medeiros, 1979)

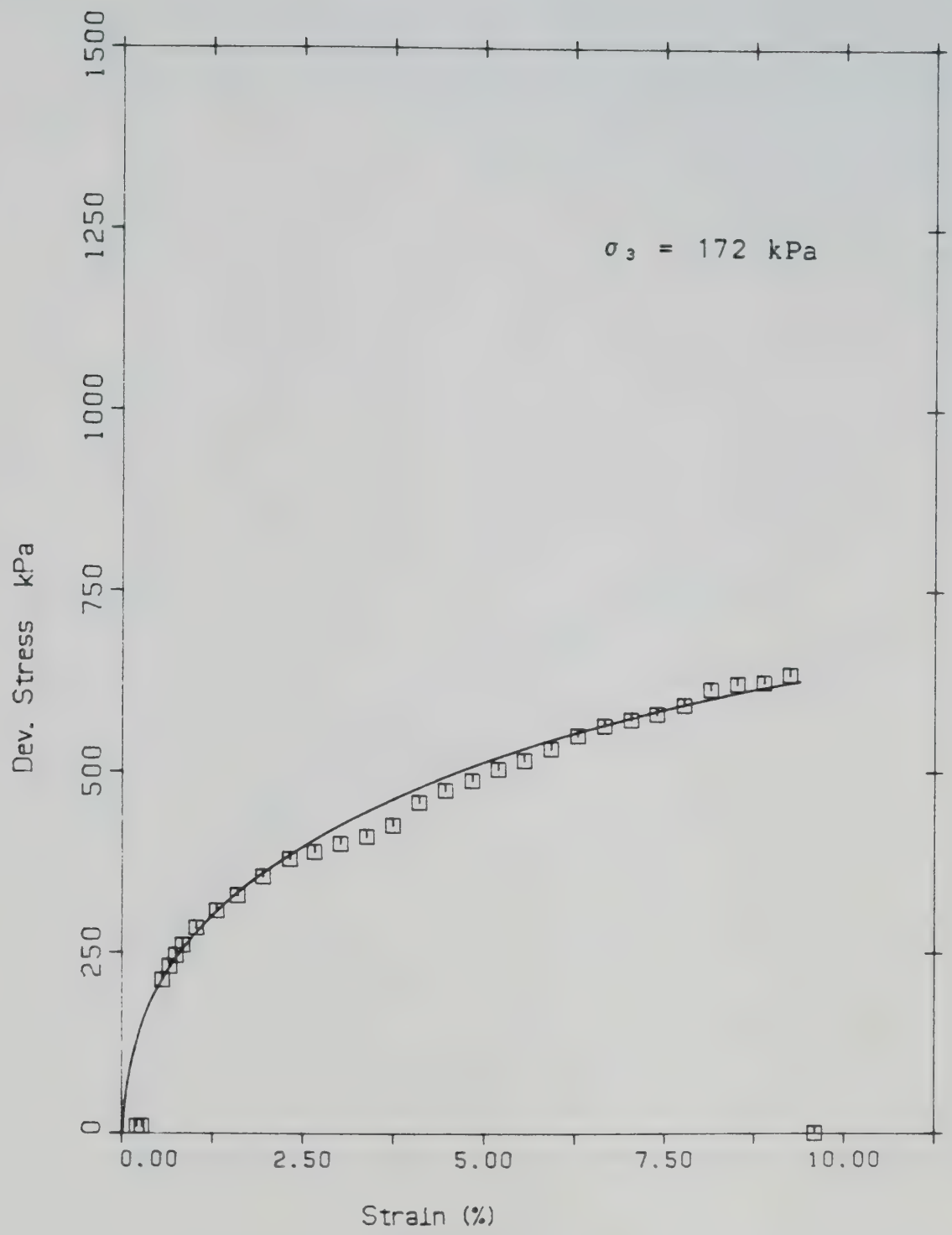


Figure 5.7 Passive compression test in till

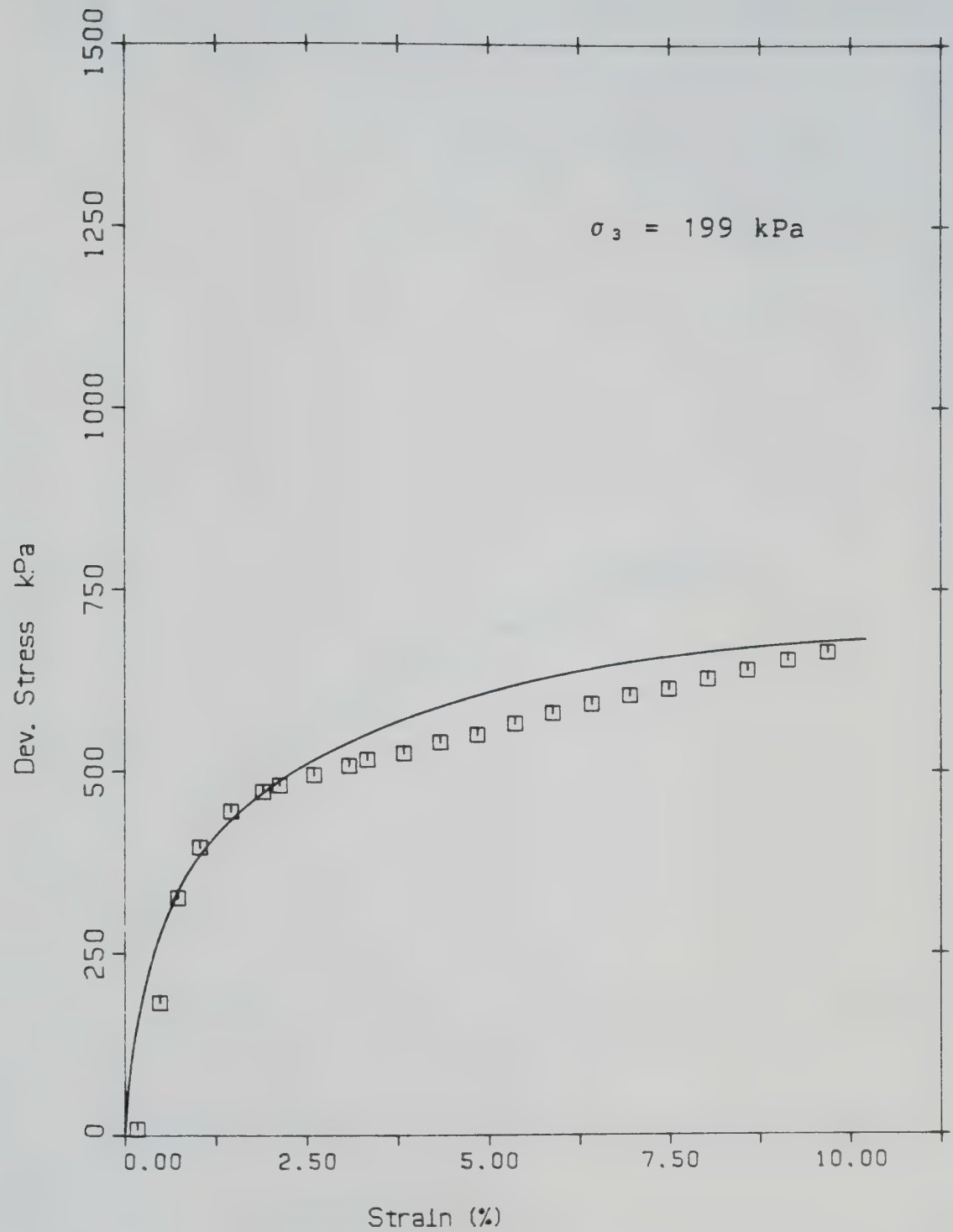


Figure 5.8 Passive compression test in till

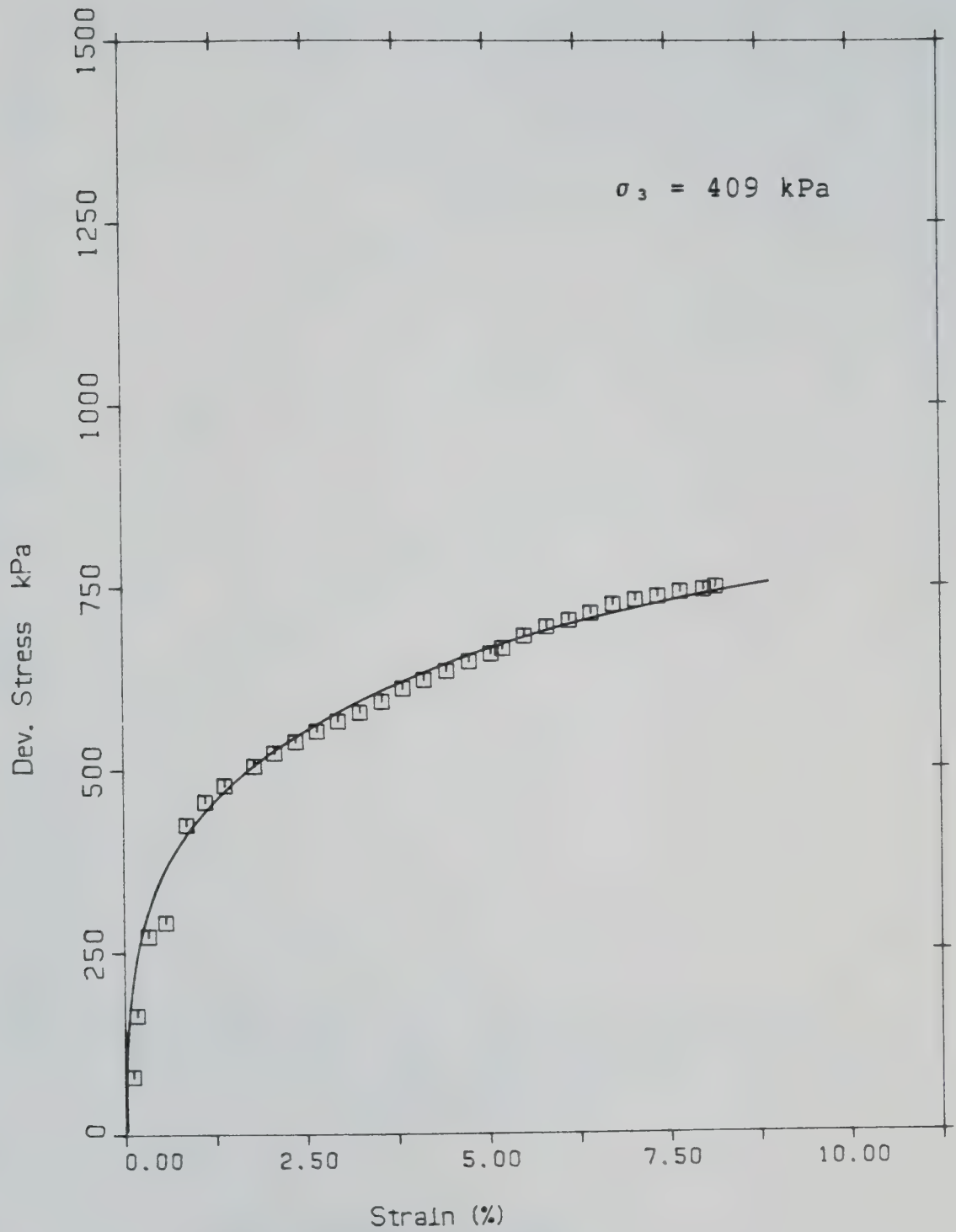
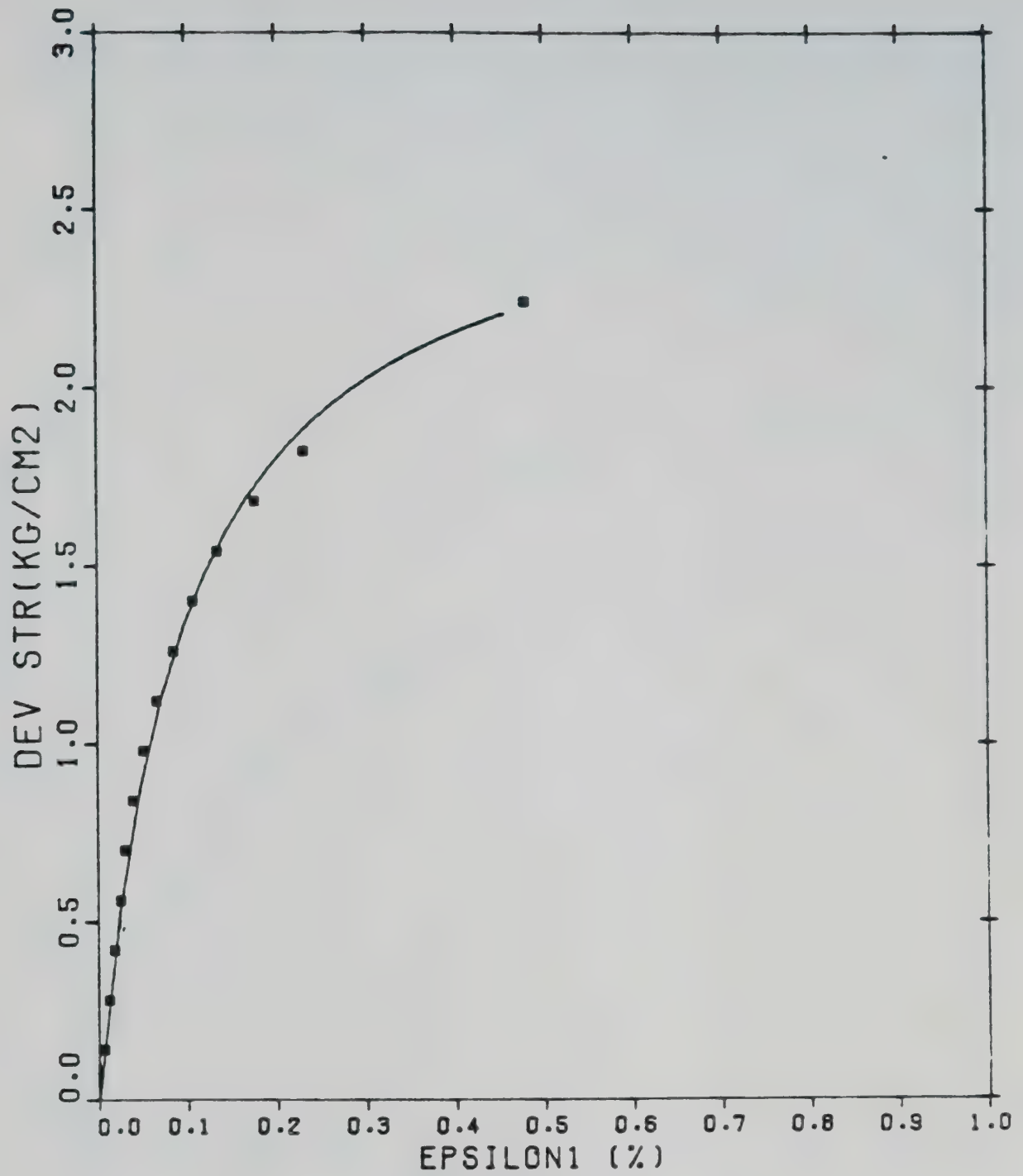
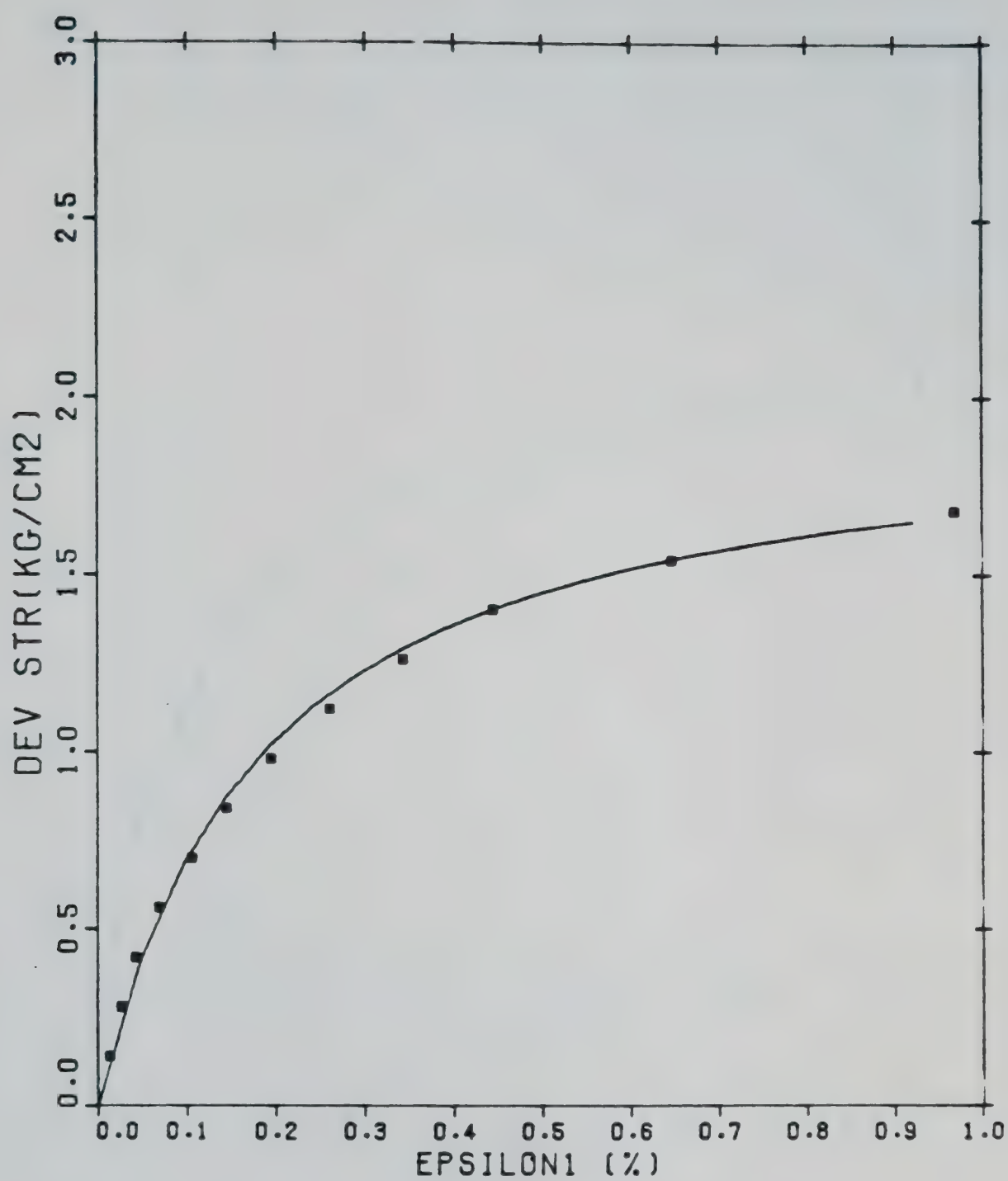


Figure 5.9 Passive compression test in till



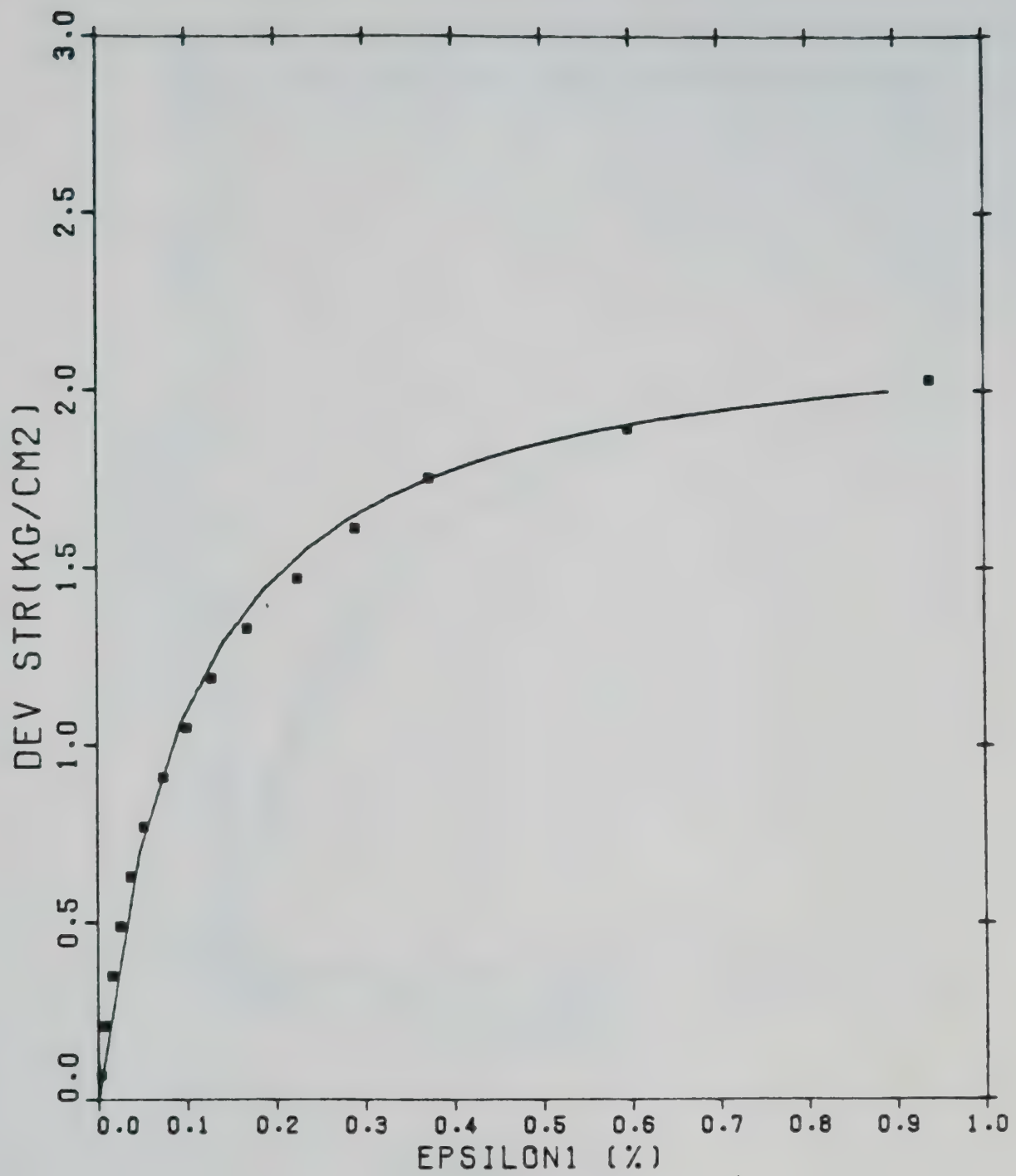
SIGMA3 2.80

Figure 5.10 Active compression test in till (After Medeiros, 1979)



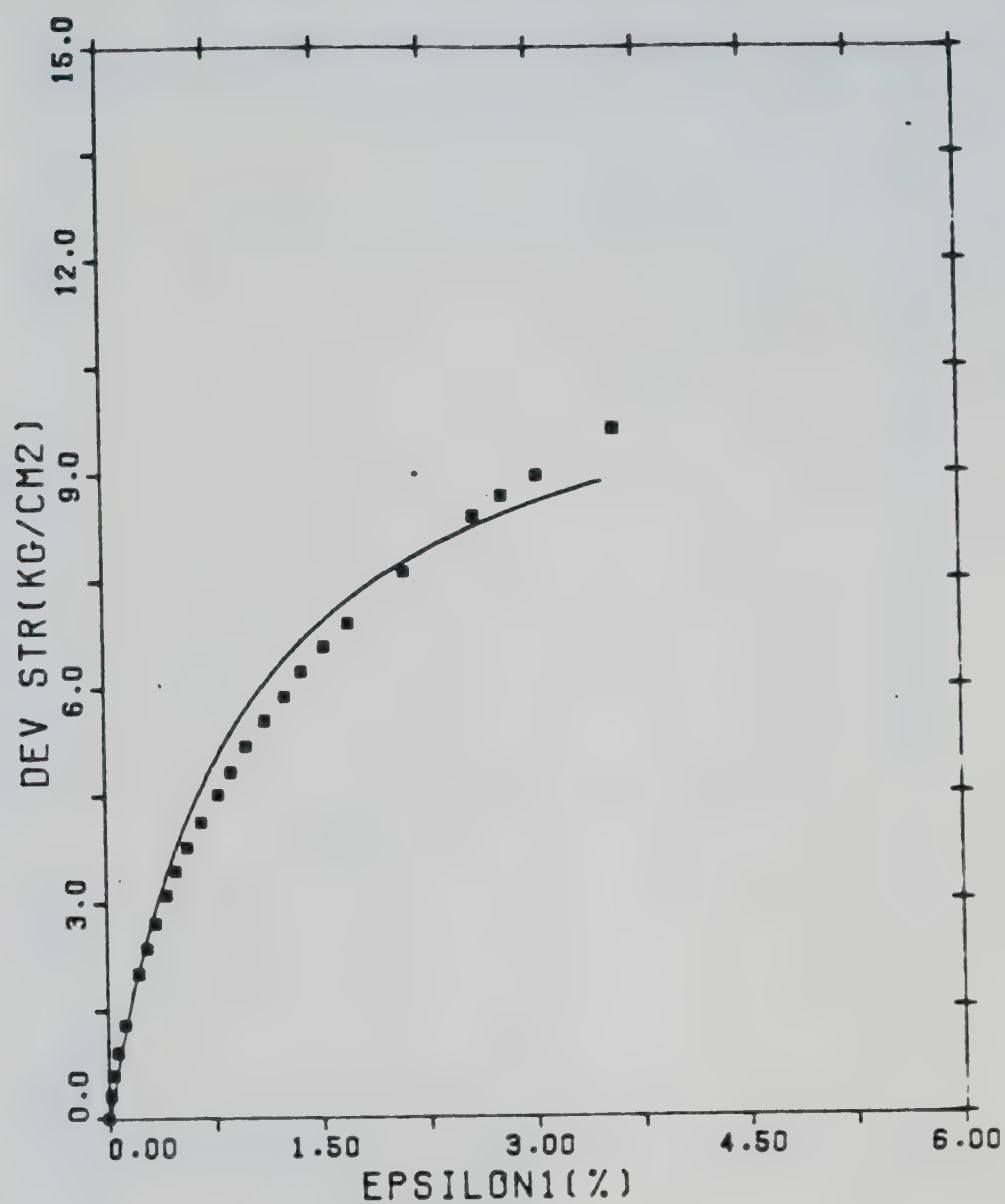
SIGMA3 2.10

Figure 5.11 Active compression test in till (After Medeiros, 1979)



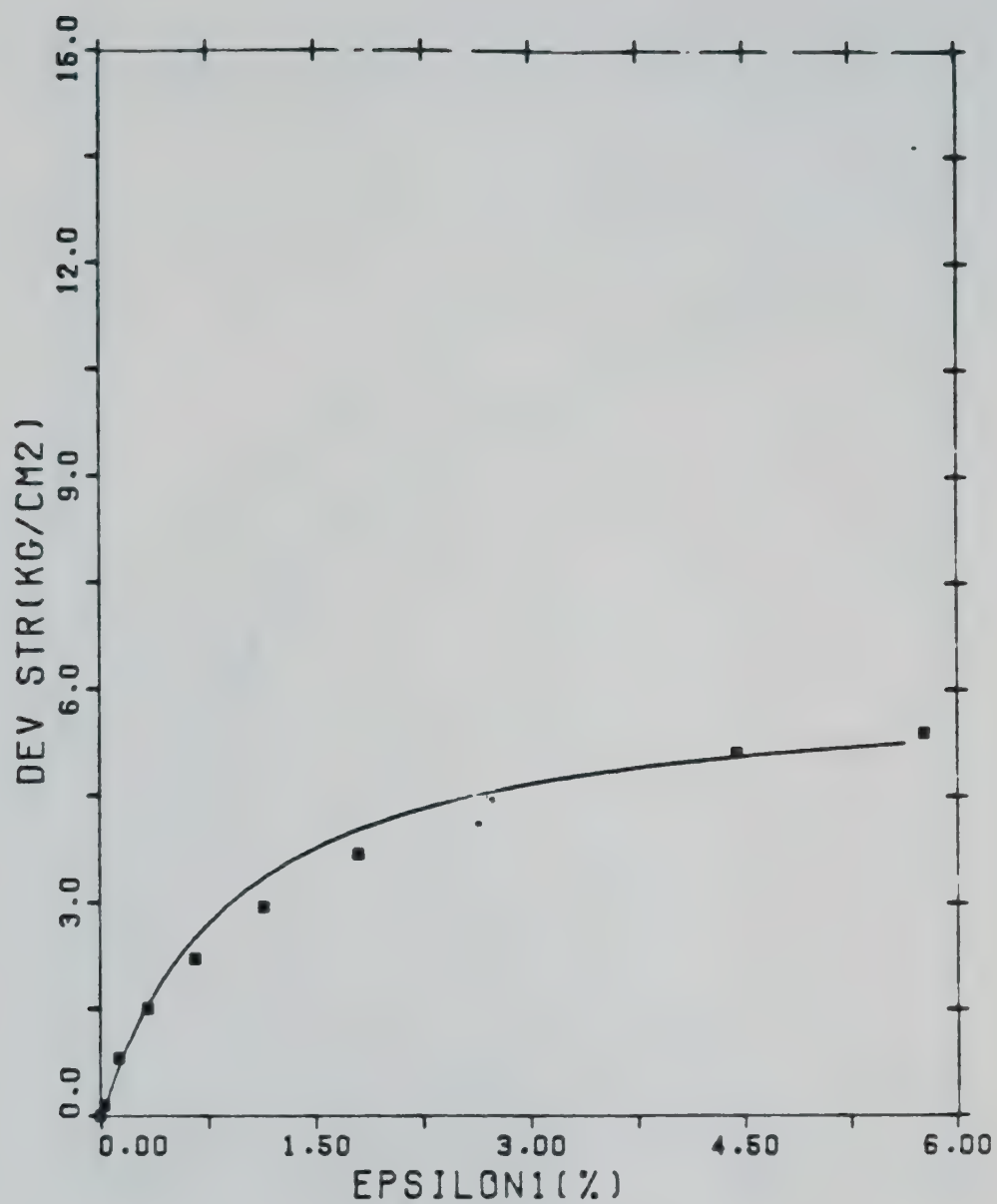
SIGMA3 2.45

Figure 5.12 Active compression test in till (After Medeiros, 1979)



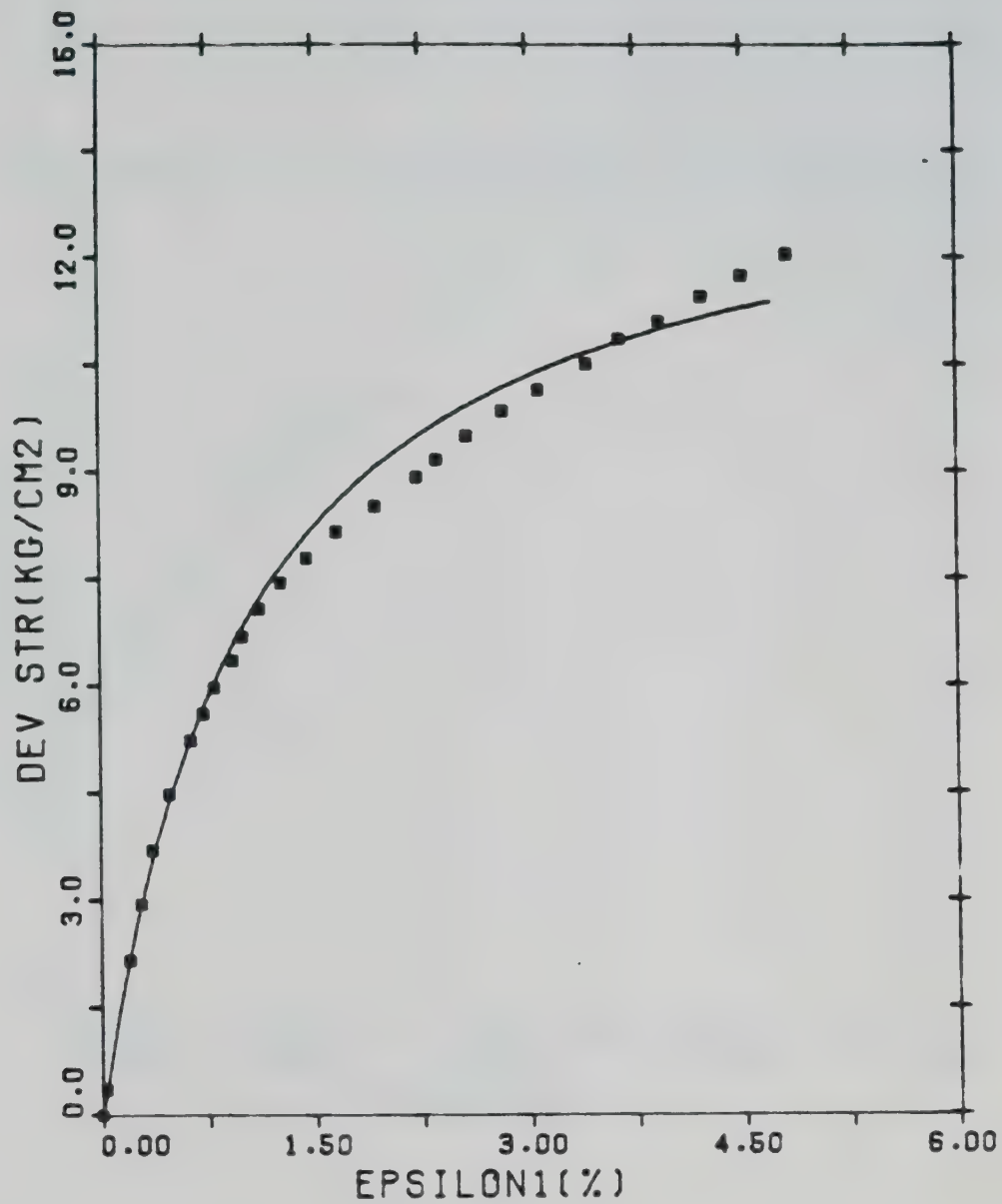
$$\text{SIG3}=1.75$$

Figure 5.13 Plane strain passive compression test in till
(After Medeiros, 1979)



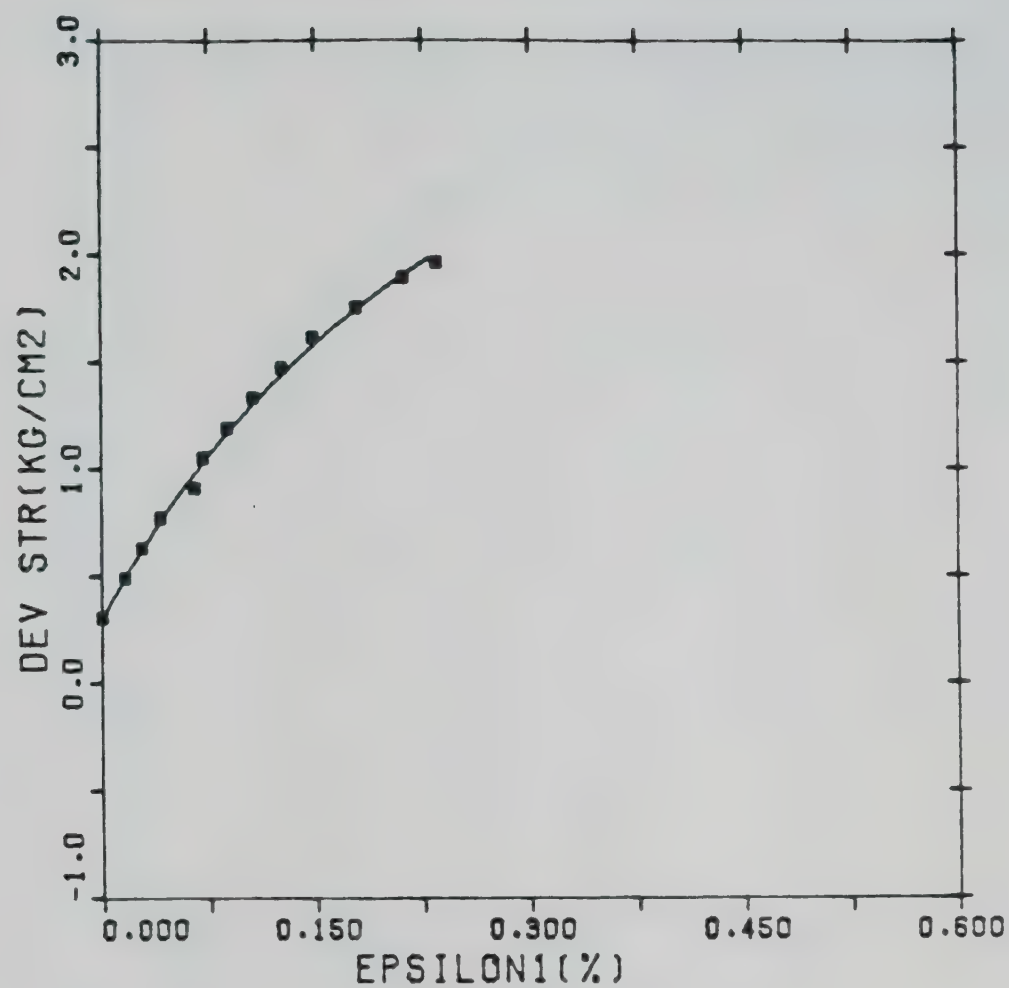
SIG3=2.10

Figure 5.14 Plane strain passive compression test in till
(After Medeiros, 1979)



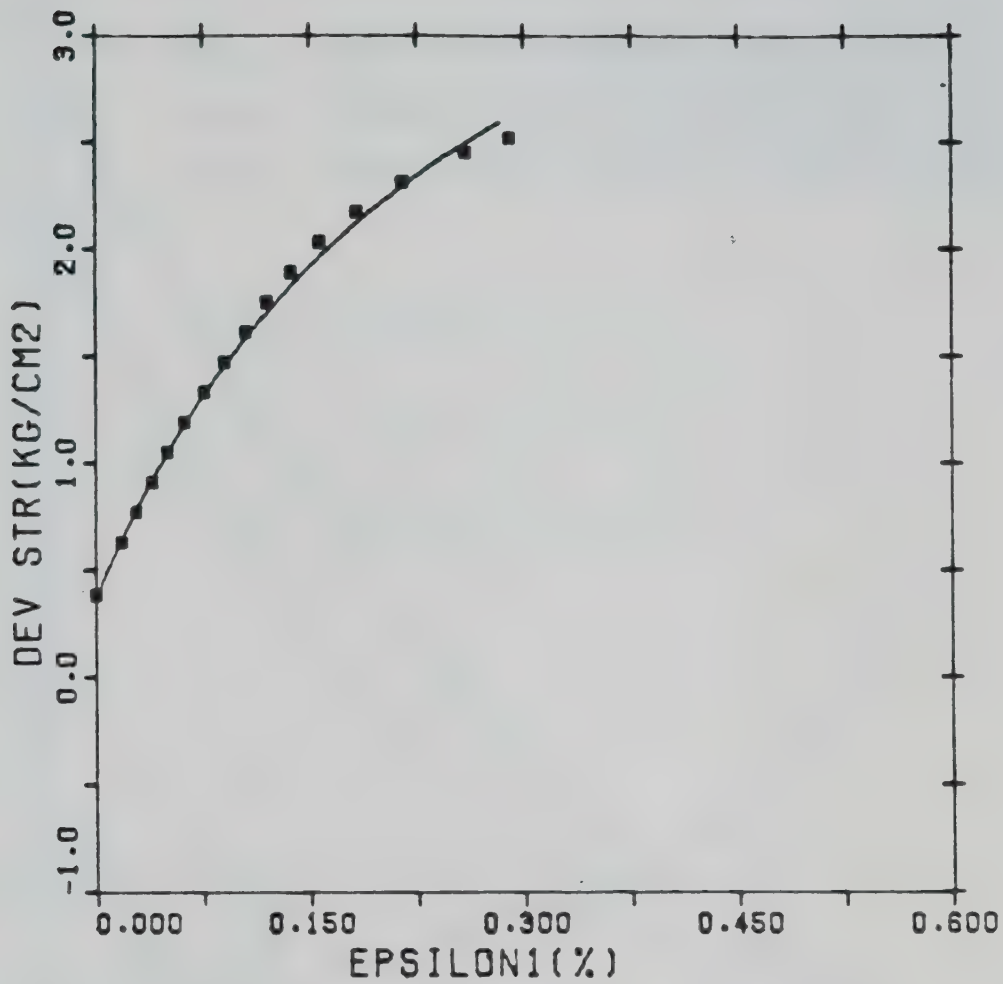
$$\text{SIG3}=2.45$$

Figure 5.15 Plane strain passive compression test in till
(After Medeiros, 1979)



$$\text{SIG3} = 1.726$$

Figure 5.16 Plane strain active extension test in till
(After Medeiros, 1979)



SIG3=2.205

Figure 5.17 Plane strain active extension test in till
(After Medeiros, 1979)

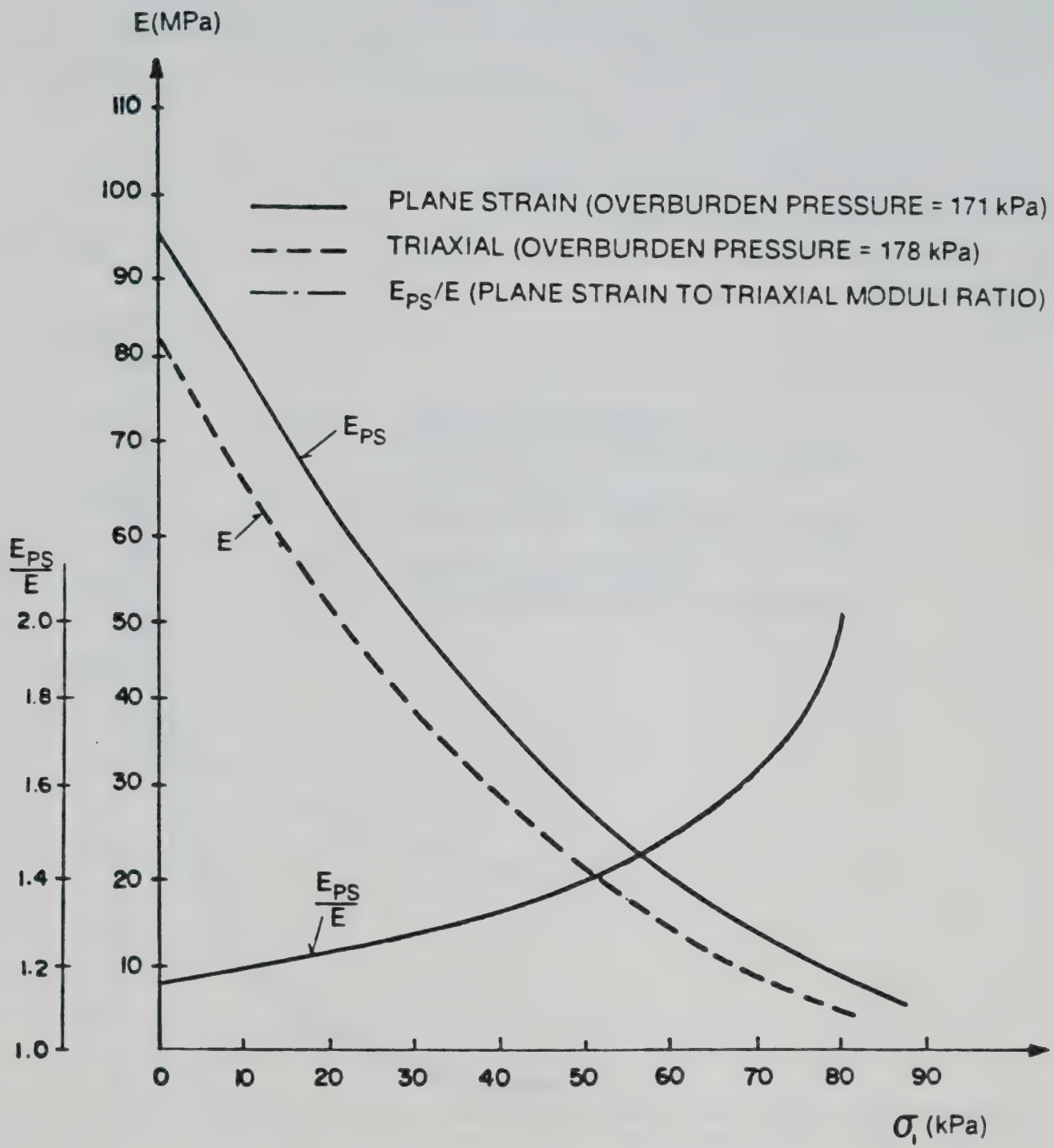


Figure 5.18 Comparison between the modulus of deformation for triaxial and plane strain passive compression tests on till (After Medeiros, 1979)

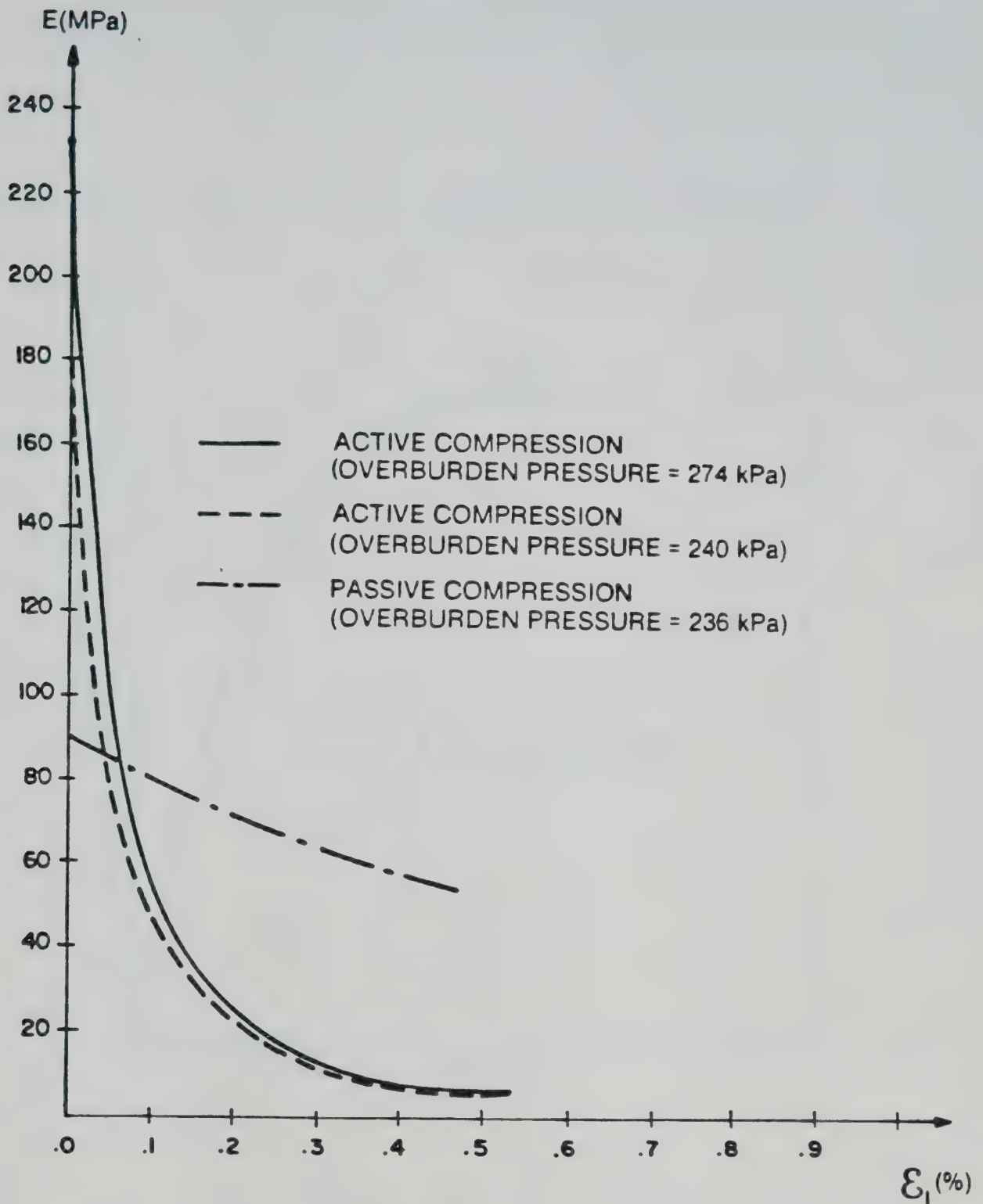


Figure 5.19 Comparison between the modulus of deformation for triaxial passive and active compression tests on till (After Medeiros, 1979)

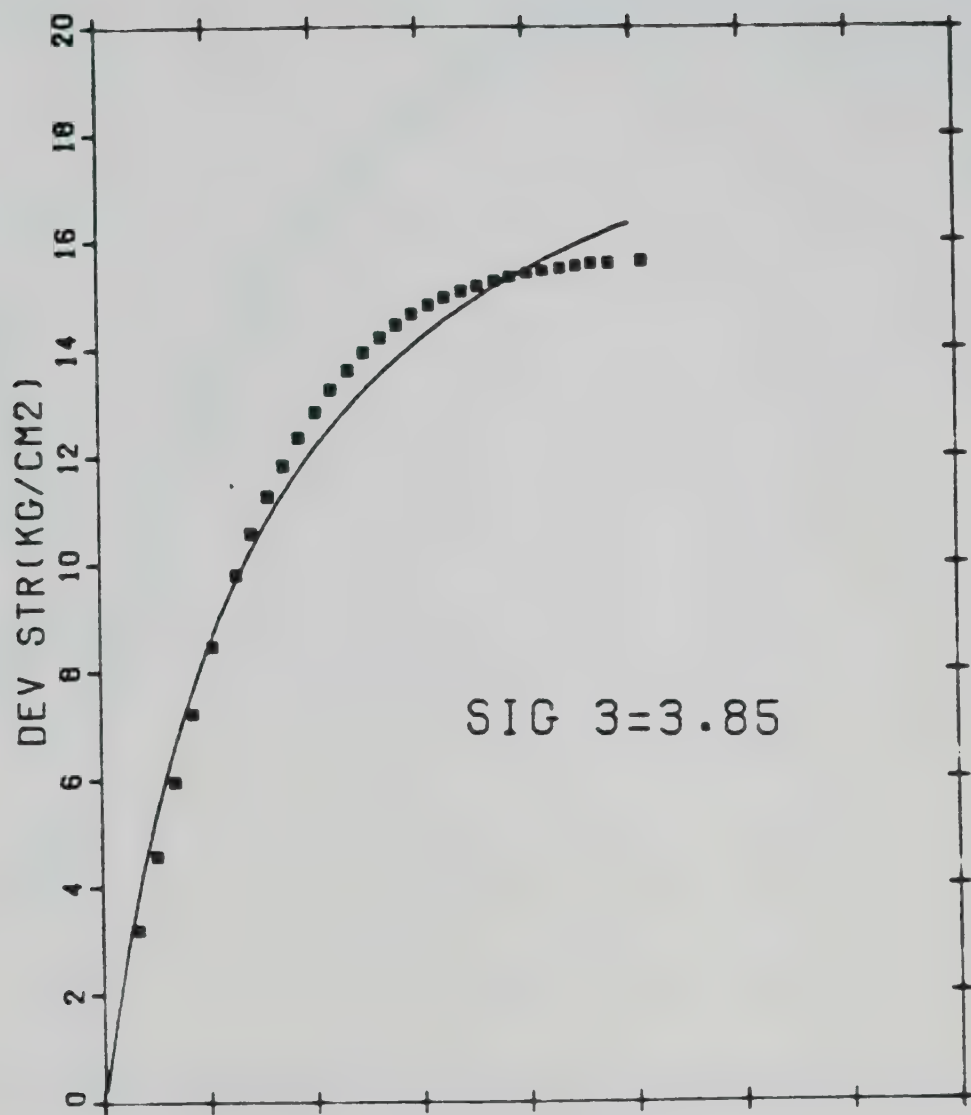
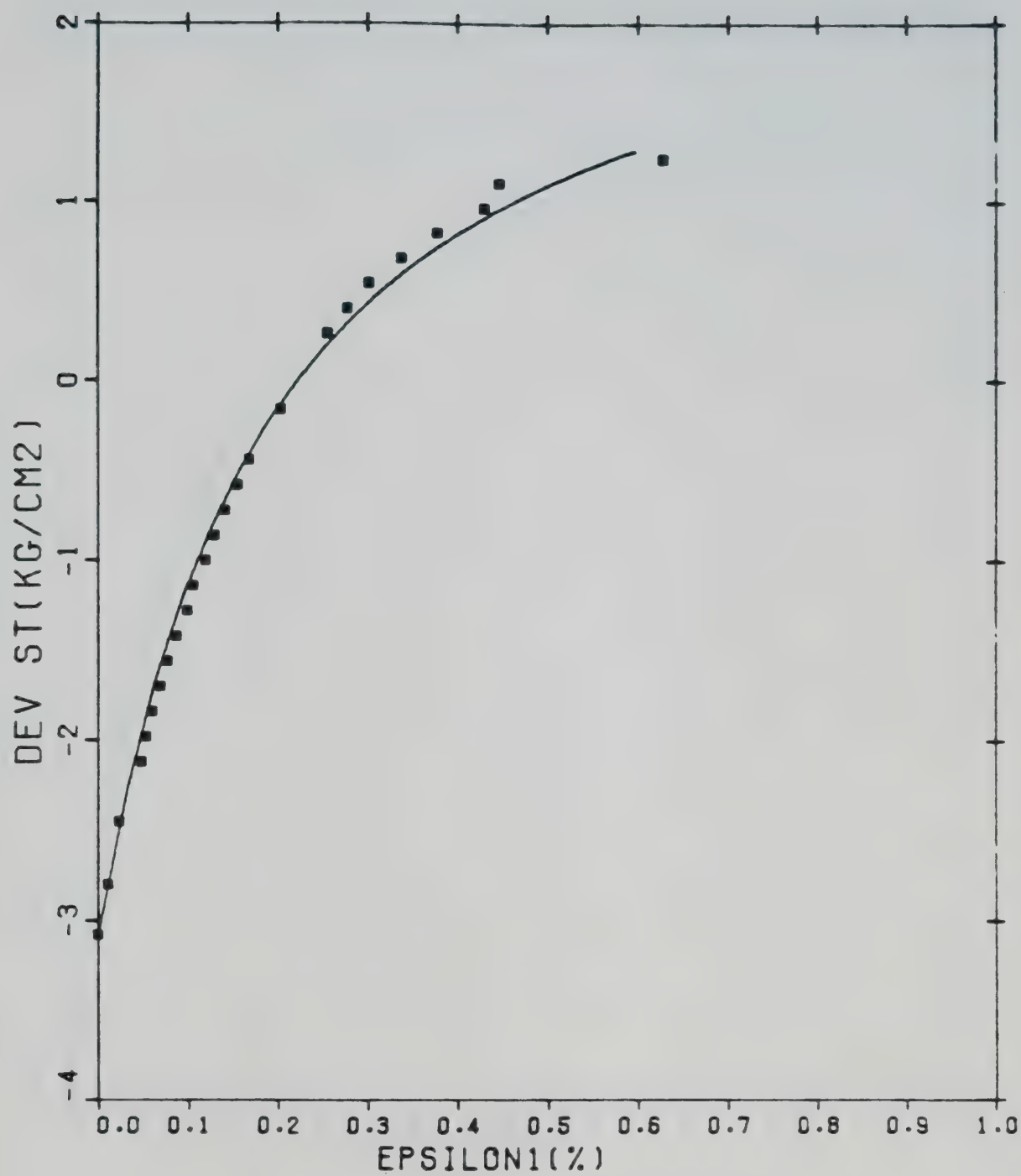
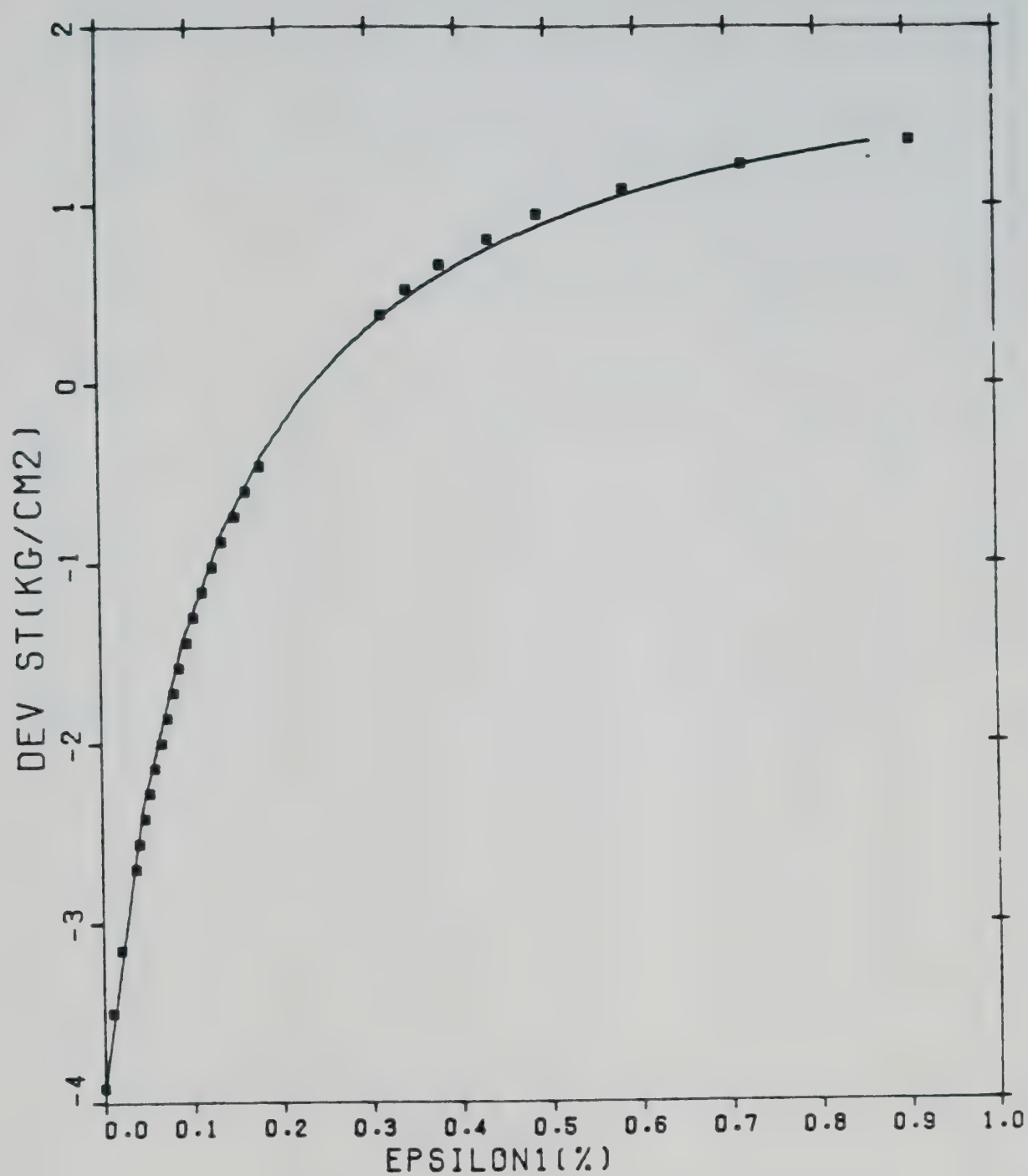


Figure 5.20 Passive compression test in sand (After Medeiros, 1979)



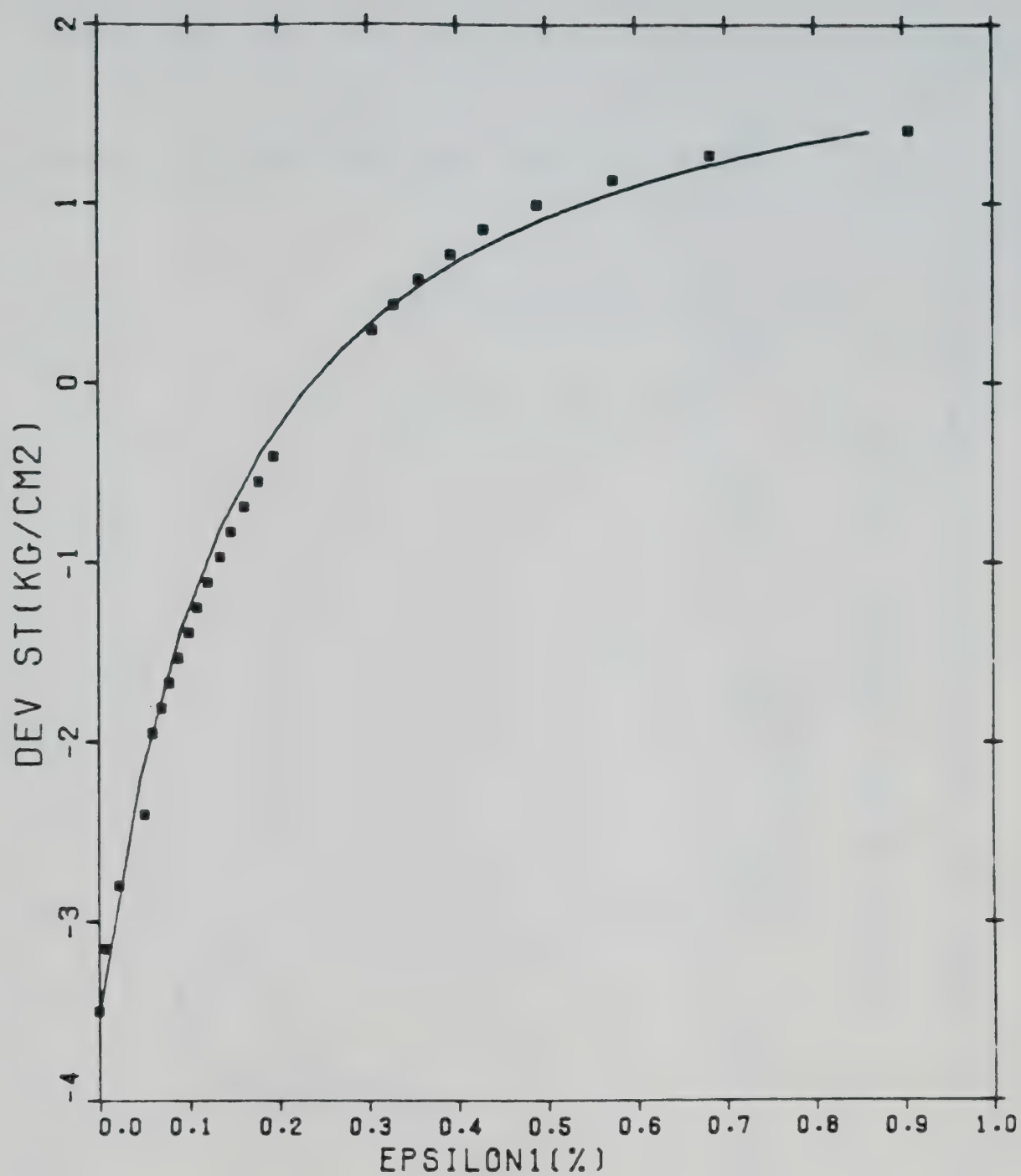
SIG3=1.733

Figure 5.21 Active compression test in sand (After Medeiros, 1979)



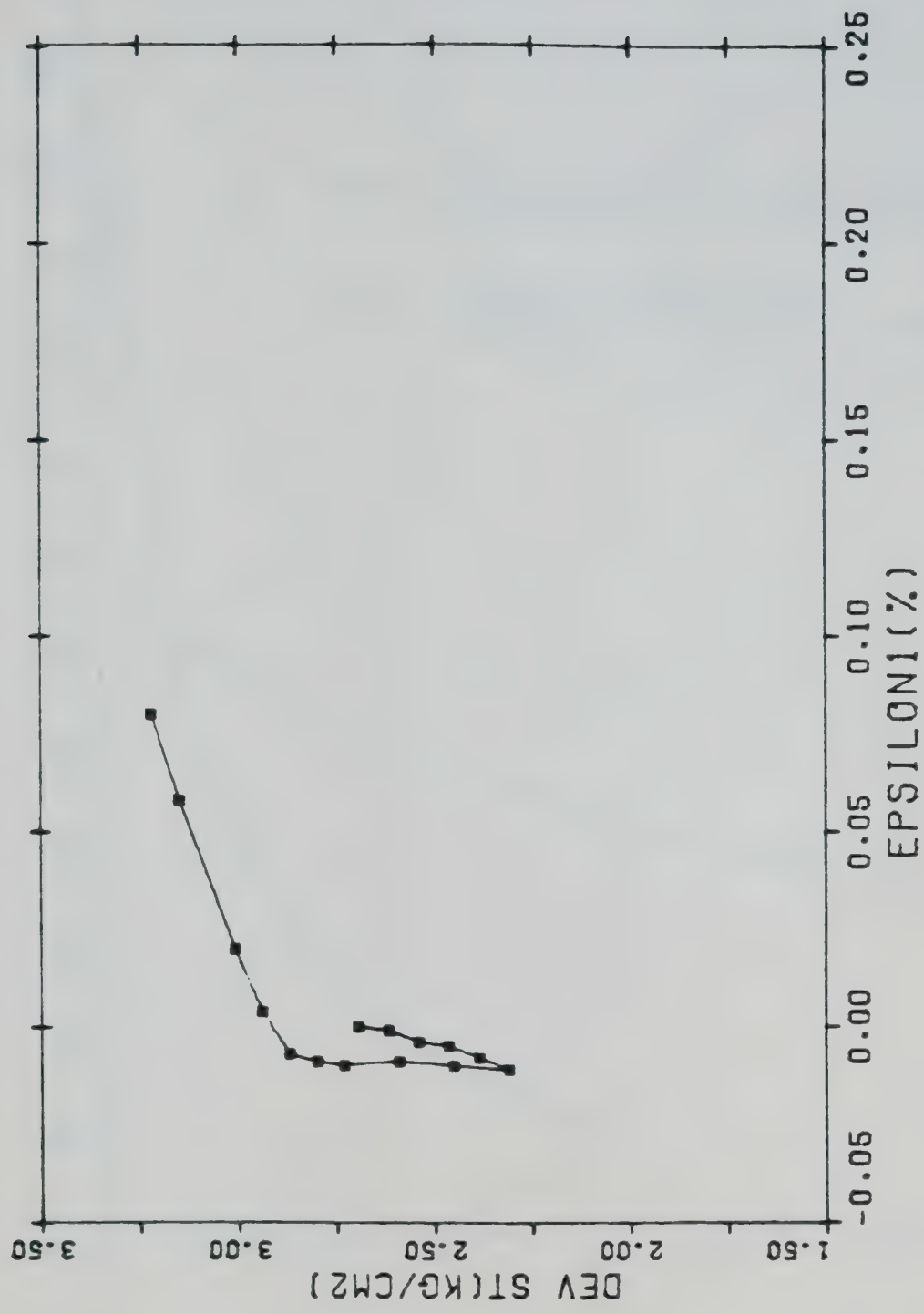
SIG3=2.205

Figure 5.22 Active compression test in sand (After Medeiros, 1979)



$$\text{SIG3} = 1.969$$

Figure 5.23 Active compression test in sand (After Medeiros, 1979)



SIGV= 4.9 SIGH= 2.21

Figure 5.24 Proportional active compression test in sand
(After Medeiros, 1979)

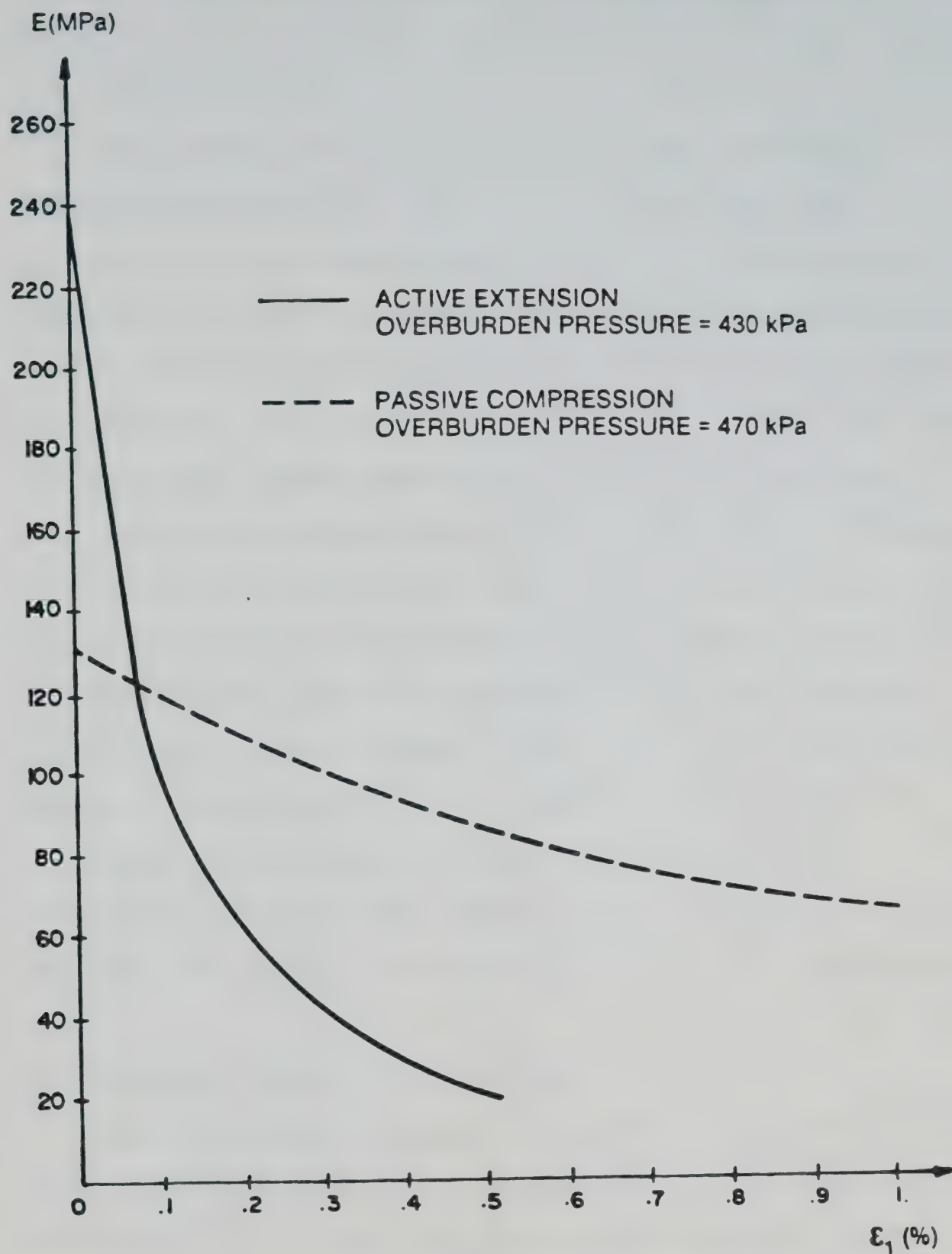


Figure 5.25 Comparison between passive compression and active extension tests on sand (After Medeiros, 1979)

6.0 Modification of the computer program

6.1 Introduction

The program used in this project was originally developed by Medeiros (1979) and has been described in Chapter 2. The program allows both linear and nonlinear analysis. For nonlinear analysis, a different approach was used in analysing the case studies. Additional functions were added to the program to perform this calculation for tie back wall as discussed in the following paragraph.

Since the program was initially developed for braced excavation, the analysis of the anchored soldier pile and lagging wall requires changes to the original program. On the other hand, due to the excavation procedure and the installation of the soldier piles, additional functions were added to the program. All the additional functions are discussed in more detail in the following section. Description of input data can be found in Appendix A. The program and a sample input data are listed in Appendix B.

6.2 Functions added to the program

As discussed in Chapter 3 the installation of the soldier pile started after the removal of 5.2 meters of soil below the ground level. In the original program, the supporting wall was present before excavation began. In order to allow the installation of the sheet pile after a certain depth of excavation, some elements properties were

changed from soil to those of the retaining wall. This change of element property was added to the original program by changing the elastic modulus and a change in thickness was also allowed to represent the equivalent stiffness of the retaining wall.

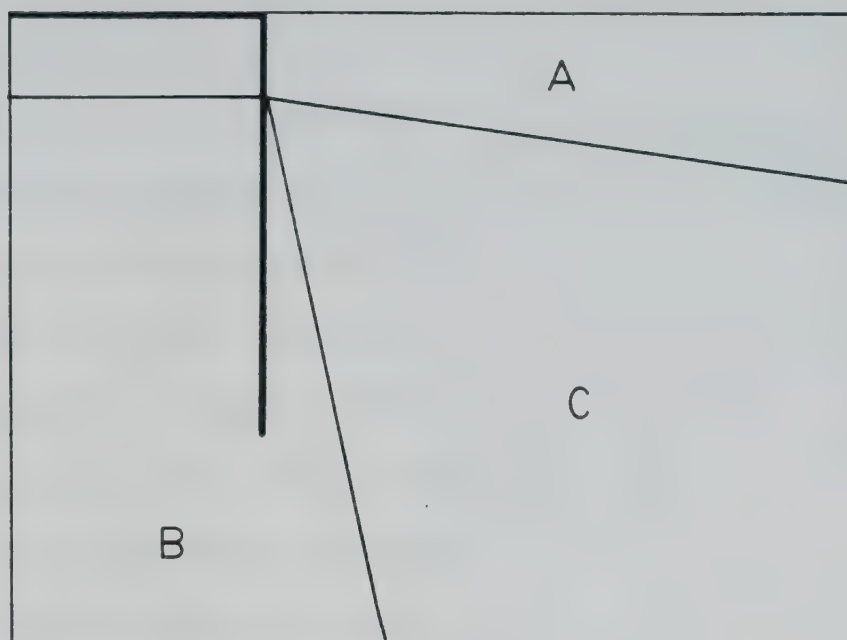
During the analysis, an assumption that the movement at the bottom of the soldier pile was restricted since the slope indicator measurements took a reference point at the bottom of the soldier pile. To prevent the movement at the bottom of the sheet pile, a change in boundary condition was allowed in the program. Changing the original boundary condition code of the appropriate nodes to the desired boundary condition code accomplished this. Besides this, the introduction of the prestressed load required another additional function inserted into the program. The inclined prestress force of the tie back was resolved into the vertical and horizontal components. These two components were then applied at the nodes located where the anchor existed. As described before the soldier pile was installed after a certain depth of excavation. The soil movements were computed from the finite element analysis due to this removal of soil. In the original program, the movements were accumulated for each phase of excavation. The slope indicators recording the horizontal deflections were installed after the installation of the sheet piles and zero readings were recorded at this level. The computed deflection from the finite element analysis for this

excavation was therefore necessarily reset to zero after the excavation was complete and the wall was installed.

For the nonlinear analysis, three different stress paths to represent the appropriate soils were used (Eisenstein and Medeiros, 1983) as described in Chapter 5. The zones remain the same throughout the analysis. In actual field condition, not only the modulus of deformation change due to excavation but also the zones that represent the soil. It was unrealistic to use plane strain active compression or triaxial active compression parameters to represent the soil from top to bottom of the excavation. On the other hand, the soil parameters would change after each step of excavation. The elements beside the vertical wall were changed from proportional active compression to active compression (plane strain or triaxial) as shown in figure 6.1.

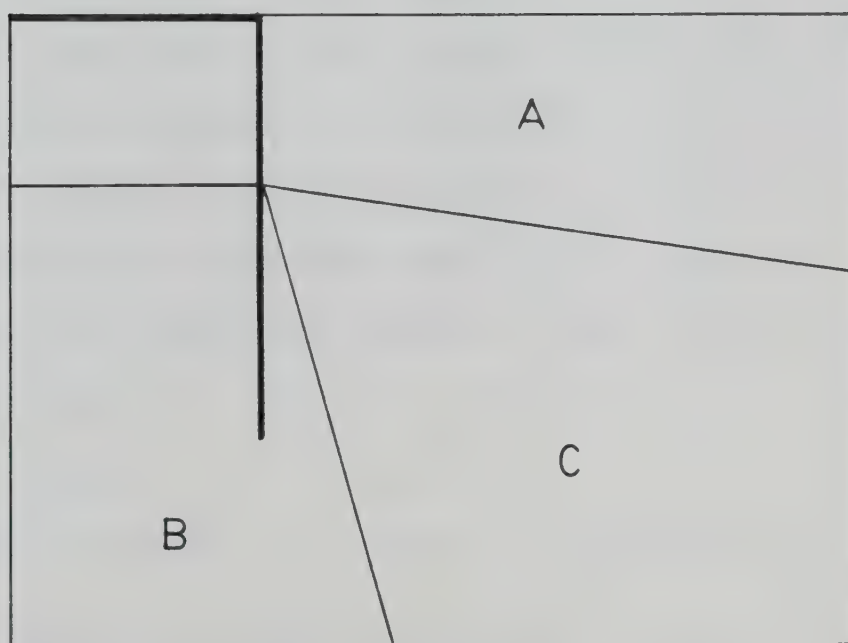
6.3 Summary

The capability of resetting to zero the soil movements was inserted in the main program. The rest of the additional functions, the installation of the retaining wall, changing of the boundary conditions, application of the prestress loads and changing of the soil properties were inserted at the end the the subroutine NLOAD of the original program (Appendix B).



PHASE 1

NOTE : ZONES A , B AND C ARE SAME IN FIG. 5.2



PHASE 2

Figure 6.1 Stress path changes due to excavation

7.0 Result of analysis

7.1 Introduction

The analysis of the case studies described in Chapter 3 were conducted by using the finite element program discussed in Chapter 6. The case studies include the excavation of three LRT stations and the excavation for the parking structure of a multistorey building located at 106 Street and 100 Avenue. In order to perform the analysis, the laboratory test data outlined in Chapter 5 were used. Finite element meshes were developed for the LRT stations and the multistorey building. Equivalent stiffness of the structural elements were determined for the finite element analysis. Assumptions used and the results of analysis are presented in later sections in this Chapter.

The stiffness of the element can be changed by varying either t (thickness) or E (modulus of elasticity) which are constant inputs into the program. A thickness of 715.6 mm was employed to represent the tangent pile wall.

Medeiros analysed the short pile section of the Churchill Station while the long pile section was analysed in this study.

The influence of adjacent structures were not significant since the weight of the soil excavated for the structures were greater than the total weight of the structure. During the analysis, no additional load was added to the retaining wall by the adjacent structures.

For the soldier pile and lagging wall at 106th Street and 100th Avenue, the stiffness of the soldier pile was greater than the timber lagging. Only the stiffness of the soldier pile was used to evaluate the thickness to obtain an equivalent stiffness of the soldier pile in the analysis. This was carried out since the slope indicator was mounted on the soil side of the soldier piles.

The actual field condition in the soldier pile wall was not plane strain. Tsui and Clough (1974) showed that deflections of the tie back soldier pile wall in three dimensional analysis differs from the results obtained from two dimensional analysis. In the case study in this project, since measurements were made in the soldier piles only, two dimensional analysis was performed to compute the average stress distribution and displacements of the soldier pile.

The computer analysis for these case records were performed under four assumptions in order to evaluate which assumption provides the best comparison between the calculated and the measured wall deformation.

1. Linear elasticity

The soil was assumed to behave as a linear elastic material throughout the analysis. The modulus of elasticity was obtained from field pressuremeter test results reported by Thurber (1982).

2. Non-linear elasticity

In each analysis the soil was represented using laboratory determined results. The test results were

input into the analysis using the hyperbolic stress strain representation discussed in Chapter 2.

(a) Triaxial passive compression

Conventional triaxial passive compression test results of the Edmonton till and the Saskatchewan sands were used to represent the soil.

(b) Triaxial active compression

Active compression test results of Edmonton till, and active extension and proportional active test results of Saskatchewan Sands were used.

(c) Plane strain active compression

Plane strain active compression test results of Edmonton till was used instead of active compression test results as described above.

The finite element configuration employed in the analysis of the LRT station excavation consists of 672 elements and 372 nodal points (figure 7.1). Because the excavations were symmetrical, only a half section was used for the computations. The analysis extends to a depth of 28 meters, which was the bottom of the boreholes, and a horizontal distance of 46 meters perpendicular to the wall was used. The boundary conditions were fixed in the horizontal direction and free to move in the vertical direction on both sides. At the bottom, both directions were fixed and the top surface was free to move in both directions (figure 7.1).

The excavation analysis was performed using a step by step or incremental analysis to simulate the actual construction sequence. The analysis proceeded with six phases of excavation for the LRT stations as outlined below and illustrated in figure 7.2:

1. Excavation of the first 2.1 meters of soil.
2. Excavation of another 1.9 meters of soil to a depth of 4 meters and installation of the first level strut (the girder) at the top.
3. Excavation to the depth of 6.5 meters (removal of another 2.5 meters of soil).
4. Excavation of another 1.2 meters of soil (7.7 meters).
5. Excavation of additional 2.8 meters of soil to reach the depth of 10.5 meters and installation of the second level of strut (the mezzanine).
6. Excavate to the final designed depth. A total 13.25 meters deep excavation for 107 Street Station and a total of 14.5 meters deep excavation for 104 Street Station.

For the Churchill station, an extra 7th phase excavation was performed to reach a depth of 15.3 meters.

The finite element mesh developed for the soldier pile and lagging wall at 106 Street and 100 Avenue consists of 497 elements and 274 nodal points as shown in figure 7.3. The size of the mesh was 28 meters wide in the horizontal direction and extends to a depth of 20 meters. The boundary conditions were as described for the LRT stations. Taking

the field excavation processes into consideration, analysis was performed in 3 phases as follow and illustrated in figure 7.4:

1. Excavation of the first 5.2 meters of soil.
2. Placement of the sheet pile. Excavation of another 2.8 meters of soil (7.6 meters deep) and installation of the tie back.
3. Excavation of the last 3.4 meters of soil to a total depth of 11 meters while only the bottom 5.8 meters was supported by the anchored wall.

In the following sections the computed wall deformations that result from using these meshes and the construction sequence outlined above will be discussed.

7.2 Horizontal movement of LRT tangent pile

The computed horizontal deflection will be compared with the field measurements and discussed. It should be noted that only elastic analysis was performed at the 104 Street Station and Churchill Station. This was based on the results obtained from a detail analysis of the 107 Street Station.

7.2.1 107 Street Station

As was discussed by Eisenstein and Medeiros (1983) for the Churchill Station, the deflection at the top of the pile was affected by the cement grout filled in the gap between the 'L' shape beam and the girder. The same stiffness ($E =$

343 MPa) suggested by them was used to represent the element of the cement grout in this analysis.

Figure 7.5 to figure 7.8 show the computed deflections of the tangent pile wall compared to the field measurements. It was observed that the prediction using the non linear elastic material parameters of the triaxial passive compression resulted in larger displacement than the field measurement (figure 7.5). The calculated displacements have a parabolic shape with a maximum deflection of 20 mm at 9 meters compared to an actual maximum deflection of 5.5 mm at 11 meters. The results from plane strain and active compression assumptions also overestimate the field deflections (figure 7.6 and 7.7). The plane strain analysis was closer to the field with a predicted maximum of 9.8 mm at about 11 meters. The comparison for the active compression showed a maximum of 12 mm at 12 meters. In both analyses the predicted curves have a parabolic shape due to the restraint at the pile tip and by the girder at the top of the tangent pile wall.

The assumption of a linear elastic soil with a modulus of elasticity ($E = 118 \text{ MPa}$) gives the best agreement with the field deformation measurements (figure 7.8). A maximum deflection of 6.2 mm was computed at a depth of about 11 meters in which the maximum of the field measurement was 5.5 mm at the same location. The calculated shape of deflection was parabolic. From the field measurements, it can be observed the deflection was affected by the street girder

and the mezzanine floor beam; the deflection at these locations were reduced due to the presence of the horizontal support and this was not observed in the computed results. Except for this deficiency, not only the magnitude of the maximum deflection was in close agreement with the field measurement but also the deflected shape of the tangent pile wall.

The computed deflection for the final phase of excavation from various assumptions are shown in figure 7.9 together with the field measurement. The overprediction of deformation by triaxial passive compression was caused by softening of the soil before laboratory testing so that the derived stress strain relationship does not represent the actual field conditions. For the plane strain and active compression assumptions, the computed deflection also overestimated the field results. This was due to the underestimate of the modulus of deformation predicted by the stress strain relationship. As test data were obtained from Medeiros (1979) in Churchill Station, it reflected that the soil in 107 Street Station was stiffer than the soil in Churchill Station. Appendix C provides a comparison of soils at the various locations along Jasper Avenue. The field measurements showed small deflections, therefore the elastic modulus of deformation should be used. Eisenstein and Morrison (1972) found that the modulus obtained from the pressuremeter was significantly higher than the comparable laboratory value. Table 7.1 shows the comparison of the

initial tangent modulus obtained from various laboratory test methods and the field measurements.

7.2.2 104 Street Station

Since the excavation of the tunnel through the 104 Street Station was carried out before the station was excavated, the applicability of stress path zones in the analysis were not the same as described in Chapter 5. The stress strain relationships were too difficult to match the field conditions so that nonlinear analysis was not performed at this station. Since the mesh was originally designed for 107 Street Station, the excavation of the tunnel was not considered. To represent the excavation of the circular tunnel, the same cross section area of elements were removed and lining was installed in rectangular shape. The elastic moduli used for the lining was calculated based on the equivalent stiffness of its liner.

From the results of the analysis of the 107 Street Station, it was found that linear elastic analysis gives the best agreement with the field deformation measurements. Therefore only an elastic analysis was performed and figure 7.10 illustrates the horizontal deflections. The comparison shows poor agreement between the field and predicted deflections. This was caused by the excavation of the tunnel for the lack of ability to track this particular construction sequence.

7.2.3 Churchill Station

The horizontal deflection resulting from using a linear elastic calculation is shown in figure 7.11. It can be observed from the curve that the predicted deflection was close to the field measurements above the base. Larger displacements below the base of the excavation are caused by restricting the soil movement towards the excavation. In actual field conditions, since short piles were presented between long piles, the soil was allowed to move freely below the short pile into the excavation. Figure 7.12 shows the deformation of both long and short pile (from Eisenstein and Medeiros, 1983).

It can be observed that the result of the elastic analysis of the long pile had a close match with the deflection above the mezzanine floor. Below the mezzanine floor, the predictions from the short pile were better since they allowed for soil movement underneath the base of the excavation. The elastic modulus for the soil in both the long and short pile analysis was 103 MPa.

7.3 Horizontal movement of the soldier pile

Figure 7.13 shows the predicted displacements of the sheet pile from the finite element analysis and the field measurements for the excavation located at 106 Street and 100 Avenue. The results computed using triaxial passive compression assumption was significantly larger than field measurements. On the other hand, the deflections predicted

from the other three assumptions (section 7.1) resulted in good agreement with the field measurements. A significant change in deflection at the location of tie back anchor was observed in the field measurements but not in the computed deflections. It may be due to the assumption in the analysis of equivalent nodal force per unit thickness to represent the prestress load. In actual field condition, the prestress anchor were loaded against the walers which were supported by the soldier piles. The field measurements show a maximum deflection of 4.5 mm which indicated a small deflection so that the soil should remain in the elastic range. Since smaller depth of excavation and fewer excavation phases were involved in the project, softening of the soil was prevented in the nonlinear analysis. Both plane strain and stress path gave good predictions when compared to the field measurements.

7.4 Vertical movement of LRT tangent pile

Field settlement measurements were made only for the 107 Street Station. The calculated vertical displacements obtained from the finite element analysis are plotted together with the field data in figure 7.14. The displacements resulting from the use of the conventional triaxial passive compression test parameters again reflect the reduced modulus of deformation obtained and over predict the displacement. On the other hand, the assumption of linear elastic results in smaller displacement by 1.6 mm.

The predictions based on active compresssion and plane strain tests were in fair agreement with the field measurements. It can be observed in the diagram that the maximum displacement computed by these assumptions agree near the wall, but underestimated the settlement at greater distance from the wall. The underprediction of settlement in the linear elastic analysis may due to the anisotropic behaviour of the soil. The elastic modulus in the vertical direction should be smaller than that in the horizontal direction since it gives good predictions in horizontal deflection.

Lo and Valle (1971) carried out a series of tests in stiff clay showed that the modulus of deformation was not affect significantly by the presence of fissure. Since the effect of fissures on the modulus of deformation in Edmonton till was not known, the modulus of deformation in the vertical direction should be found from field experiments.

7.5 Lateral load and stress acting on tangent pile wall

In the 107 Street Station, load cells were installed at the mezzanine level. The normal stress computed using the various assumptions and the load cells measurements were presented in Table 7.2. All the computed stresses were lower than the field measurements. This occurred because when casting the mezzanine floor, the stress from the soil did not fully develope and therefore the mezzanine strut picked up additional force besides the force developed from below

the mezzanine floor.

Lateral stress distributions were calculated by averaging the element stresses above and below the nodes along the pile and are presented in figure 7.15. The diagrams showed the lateral stress distributions for those stress strain assumptions used compared with the apparent lateral stress diagram of Terzaghi and Peck (1967). As it can be observed the lateral stress distribution predicted from the various assumptions were not significantly different and fall in between the active earth pressure ($0.22\gamma H$) and the at rest earth pressure ($0.85\gamma H$). A reduction of lateral stress in regions where the retaining structure yields and a significant increase on the supporting struts were not observed from the calculations. Below the base of the excavation, the stresses increased rapidly to the at rest state of stress.

The lateral stress distribution for the 104 Street Station and the Churchill Station are shown in figure 7.16 and 7.17 respectively. The stress distribution between the mezzanine floor and the girder were close to the earth pressure at rest condition at the Churchill Station. In the middle between the mezzanine floor and the base of the excavation, a decrease in stress was observed. At the 104 Street Station, the stress distribution at the upper portion near the girder was close to the earth pressure at rest; then the pressure distribution dropped and increased with depth at approximately at a slope equal to 0.5.

7.6 Lateral load and stress in 106 Street and 100 Avenue

The lateral stress distributions calculated from the assumed stress strain material behaviour along with the apparent lateral stress diagram of Peck (1969) are shown in figure 7.18. The stress distribution as shown in the diagram was not affect by the assumed stress strain relationship of the soil. At the point of the tie back anchor, a higher stress resulted. The stress distribution approximated the active state of stress and increased to the at rest state of stress below the base of the excavation.

7.7 Comparison between flexible and semi-rigid wall

Except for the assumption from conventional triaxial passive compression, the stress distributions predicted from the LRT tangent pile wall showed they all fall in between the at rest state and the active state with almost a linear increase with depth. Both the field and the computed deflection shape of the tangent pile were smooth from the top to bottom. Linear elastic analyses match the field deflections. The horizontal support of the strut did not affect the shape of the calculated pressure distribution. No significant reduction in stress between the struts and stress concentration near the supporting strut were observed in any of the computed stress distribution. It was quite reasonable that almost no arching effect can be observed due to the high rigidity of the tangent pile wall.

Although the computed deflection shape of the soldier pile did not match the field measurement near the tie back location, the predicted stress distribution showed a stress concentration at this location. Apart from the tie back, a decrease in stress gradient was observed just below the anchor. It was not surprising that the arching effect occurs because of the flexibility of the wall. In actual field condition the tie back acts as a concentrated point load at the whaler which was supported by the soldier piles. The computer program can only permit an equivalent force per unit thickness in the analysis. Therefore, the actual field stress near the tie back anchor would have a value higher than that predicted from the analysis.

In comparing the rigidity of the structural retaining walls analysed, there was not much difference in the shape of deflection curves. The effect of the tie back was to reduce the displacement of the whole wall but not as much as was observed in the field measurements. This discrepancy may be compensated for by using a three dimensional analysis which could simulate the actual field condition more closely than varying the elements to suit the analysis in a two dimensional analysis. The two dimensional analysis could not account for the difference in stiffness between the lagging and the soldier pile. As a result, the stress distribution from the analysis can only be considered as an average distribution. On the other hand, the case of the tangent pile wall was different due to its high rigidity and

geometric configuration as described earlier in this section. The stiffness of the tangent pile wall was approximately sixty times the stiffness of the soldier pile wall. Besides the deflected shape, the magnitude of stress in the tie back wall was lower than that in the tangent pile wall by 40%.

In order to compare the stress distribution of the retaining walls analysed, computed stress divided by γH (stress ratio) verses depth divided by the excavation height, H (depth ratio) were plotted in figure 7.19 to 7.21. A comparison of results for the various assumptions at the 107 Street Station as shown in figure 7.19 indicated an increase in stress distribution with increasing excavation depth. The results from nonlinear analysis show almost the same distribution except near the base of the excavation. In the elastic analysis, the stresses calculated were smaller than in the nonlinear analysis. Figure 7.20 shows the results for different assumptions at the 106 Street and 100 Avenue test site. Figure 7.21 shows the comparison of all field cases for the linear elastic analysis. The coefficient for 107 Street Station increase almost linearly to a value of 0.68 at the base of the excavation. At the 104 Street Station, the coefficient drops begin at a depth ratio of about 0.3 and increase at a depth ratio of 0.45 but with a smaller coefficient than at the 107 Street Station. For the Churchill Station, the stress ratio was the highest for all case studies within a depth ratio of about 0.5; it then

dropped to a stress ratio of 0.2 at a depth ratio of 0.7 and increased to 0.5 at the bottom of the excavation. The stress ratio observed in the soldier pile was the smallest. This low stress ratio was due to the characteristic of the flexibility of the soldier pile compared to the tangent piles. Figure 7.21 illustrates this finding.

As discussed earlier, the stress in the tie back soldier pile wall was 40% lower than in the tangent pile wall which shows a significant reduction in stress due to the reduction in wall stiffness.

The maximum computed deflection obtained at the 107 Street Station was 6.1 mm for the elastic assumption. The ratio of maximum deflection to the total excavation depth calculated was 0.00046. A value of 0.00041 was obtained at the 106 Street and 100 Avenue. The stiffness of the wall did not significantly affect the deflection and this is in agreement with Egger (1972) as was discussed in Chapter 2.

Type of test	Initial tangent modulus (MPa)
Linear elastic 107 St Station 104 St Station Soldier Pile	118 147 103
Active Compression	106
Plane Strain	123
Passive Compression	43

Table 7.1 Comparison of initial tangent modulus of various tests

CONDITION	NORMAL STRESS (MPa)
FIELD MEASUREMENTS	9.70
LINEAR ELASTIC	6.49
TRIAXIAL PASSIVE COMPRESSION	8.01
STRESS PATH TRIAXIAL	9.20
STRESS PATH PLANE STRAIN	9.57

Table 7.2 Loads in mezzanine

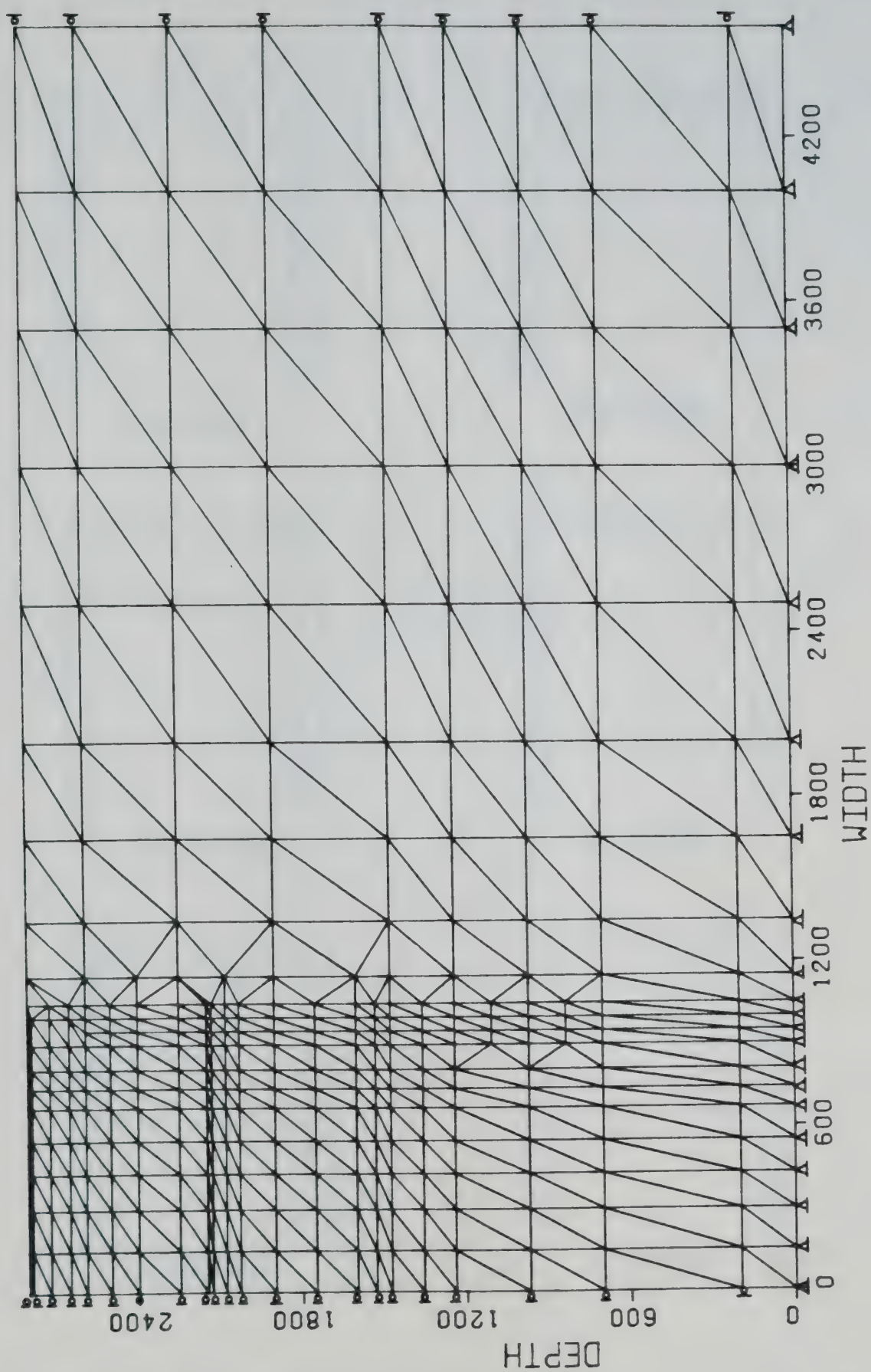


Figure 7.1 Finite element mesh for the LRT Stations

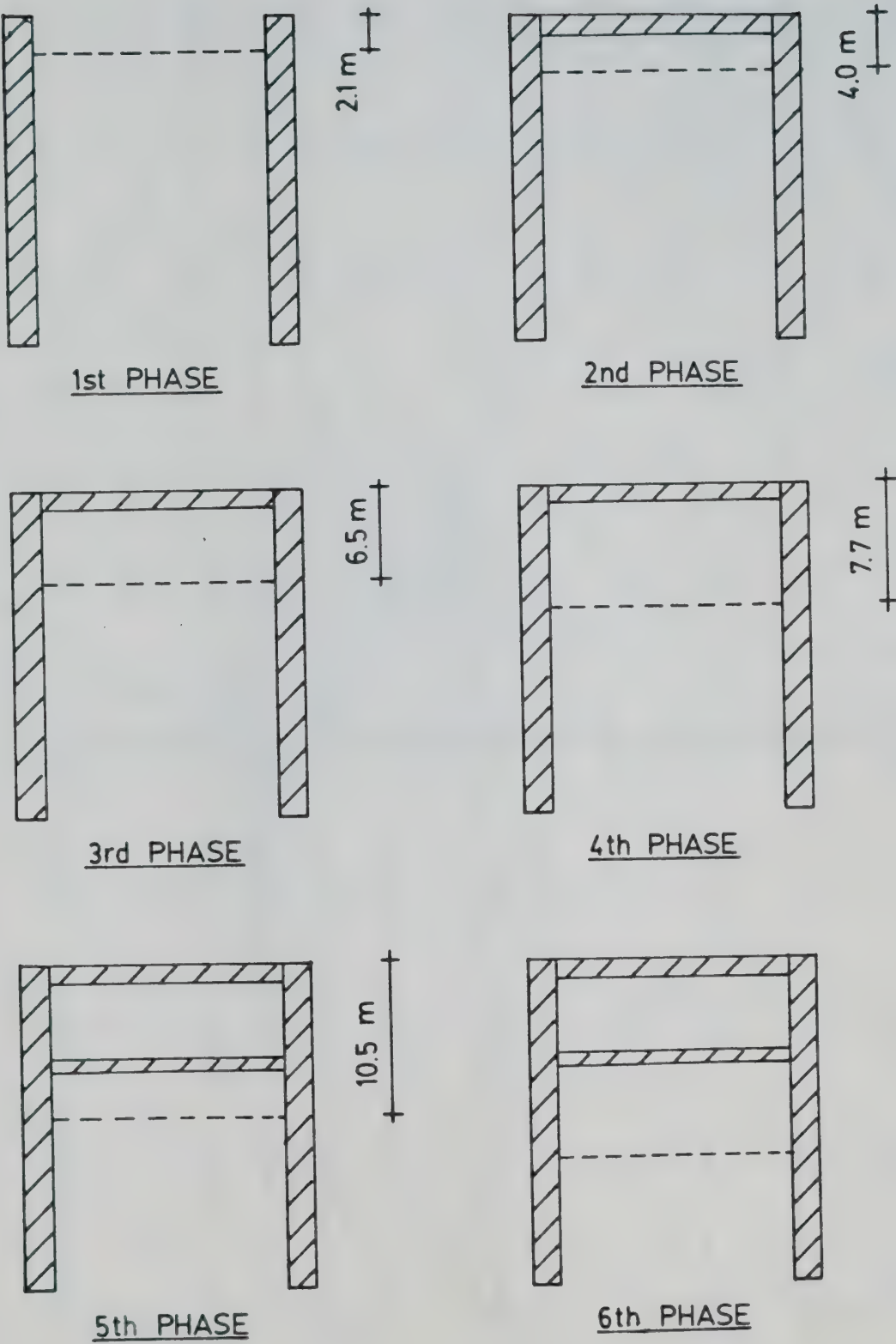


Figure 7.2 Excavation procedures for the LRT Stations

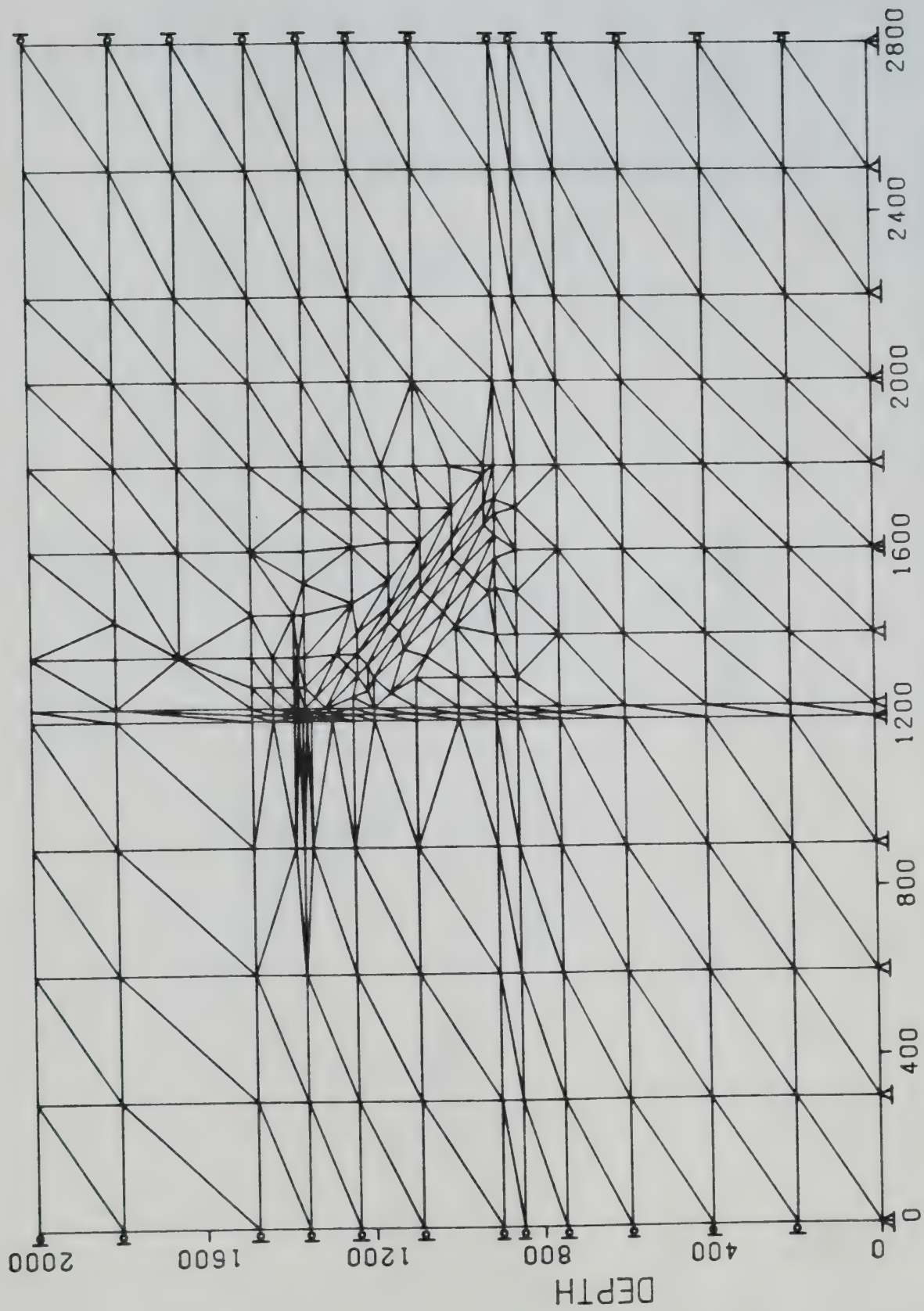


Figure 7.3 Finite element mesh for the 106 Street and 100

Avenue

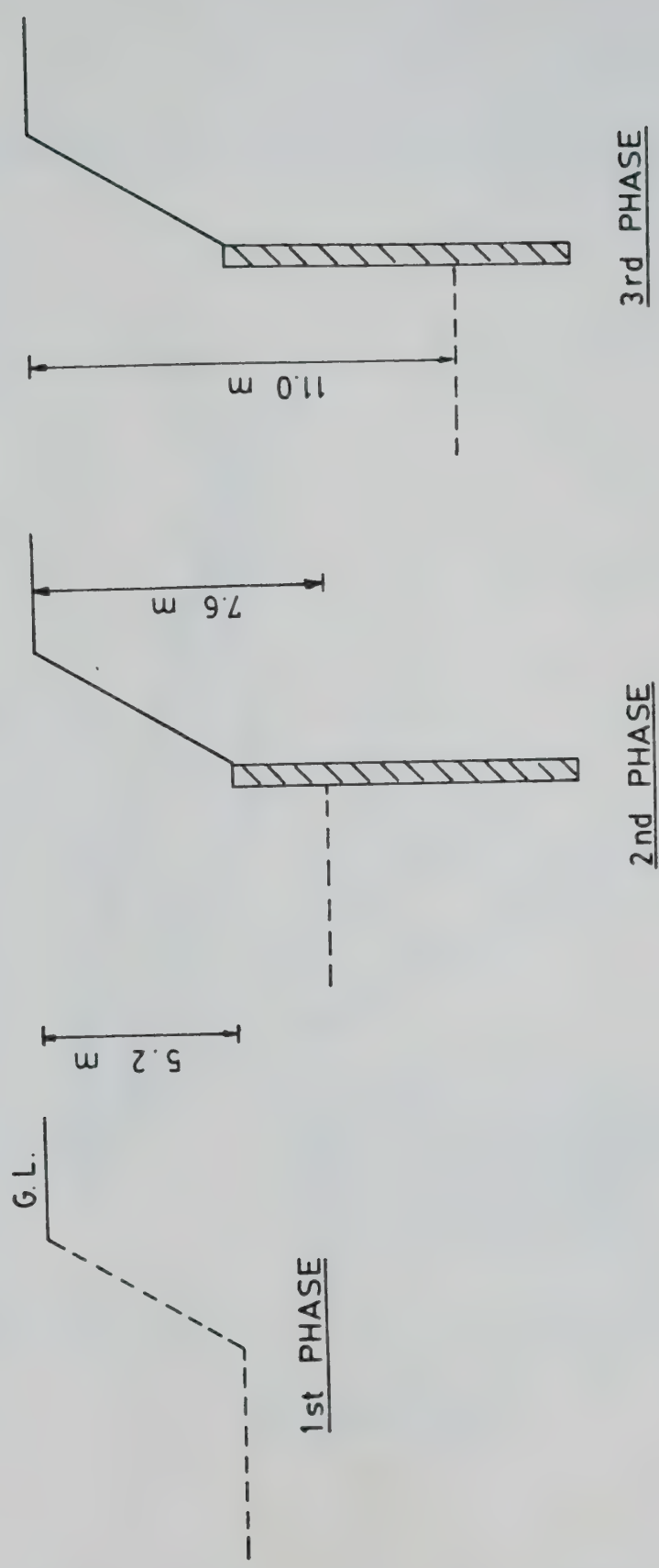


Figure 7.4 Excavation procedures for 106 Street and 100 Avenue

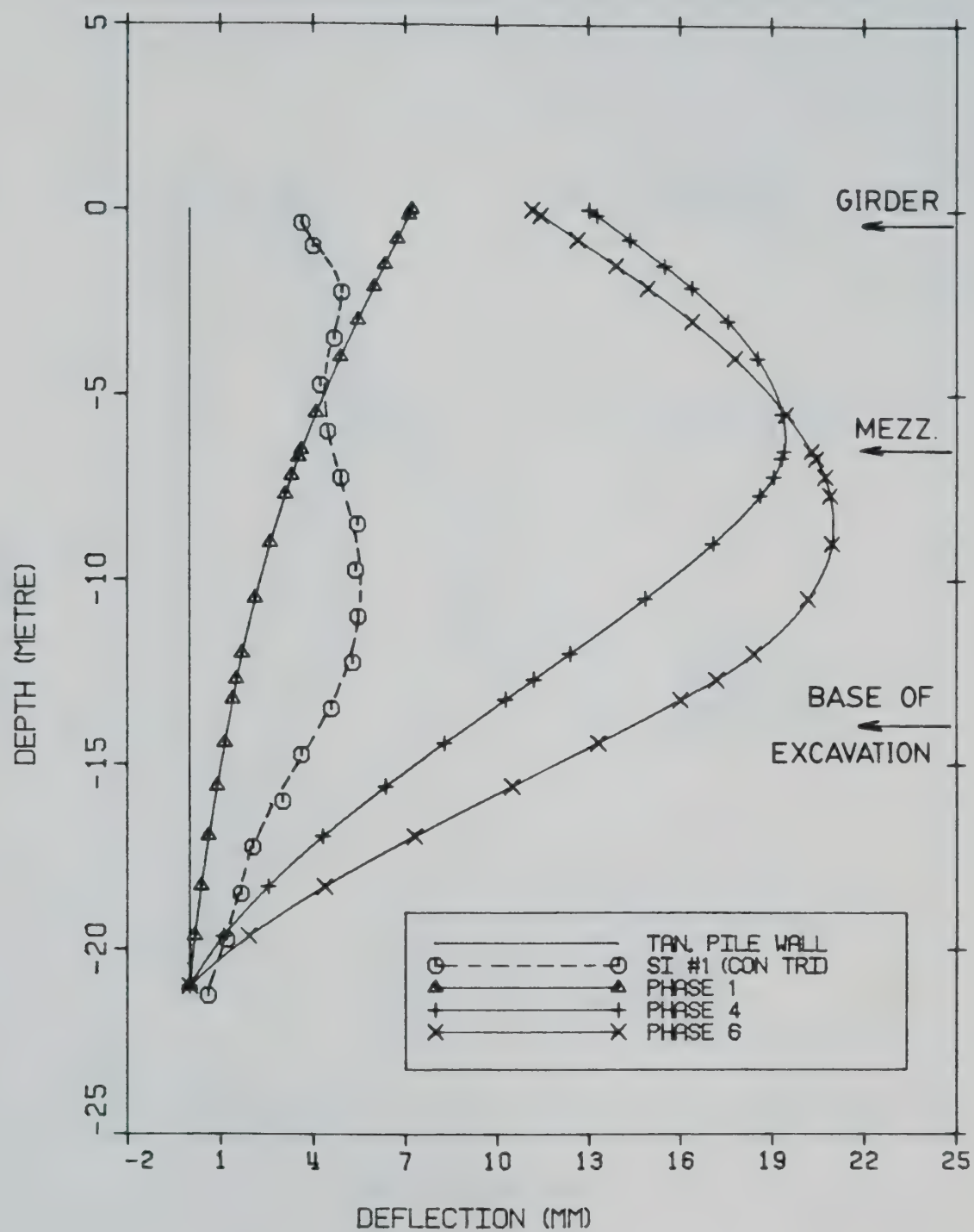


Figure 7.5 Computed deflections and field measurements of 107 Street Station, triaxial

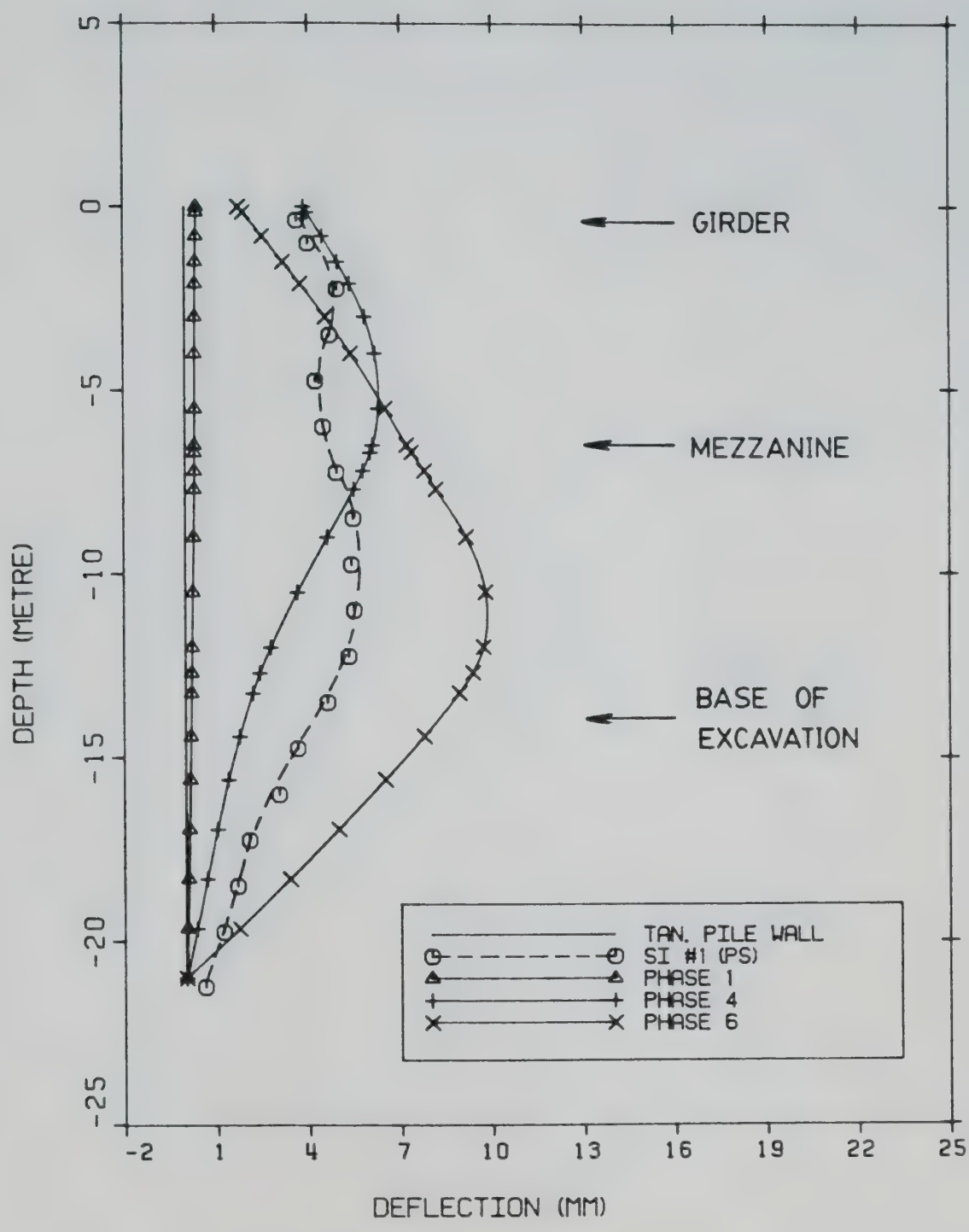


Figure 7.6 Computed deflections and field measurements of 107 Street Station, plane strain

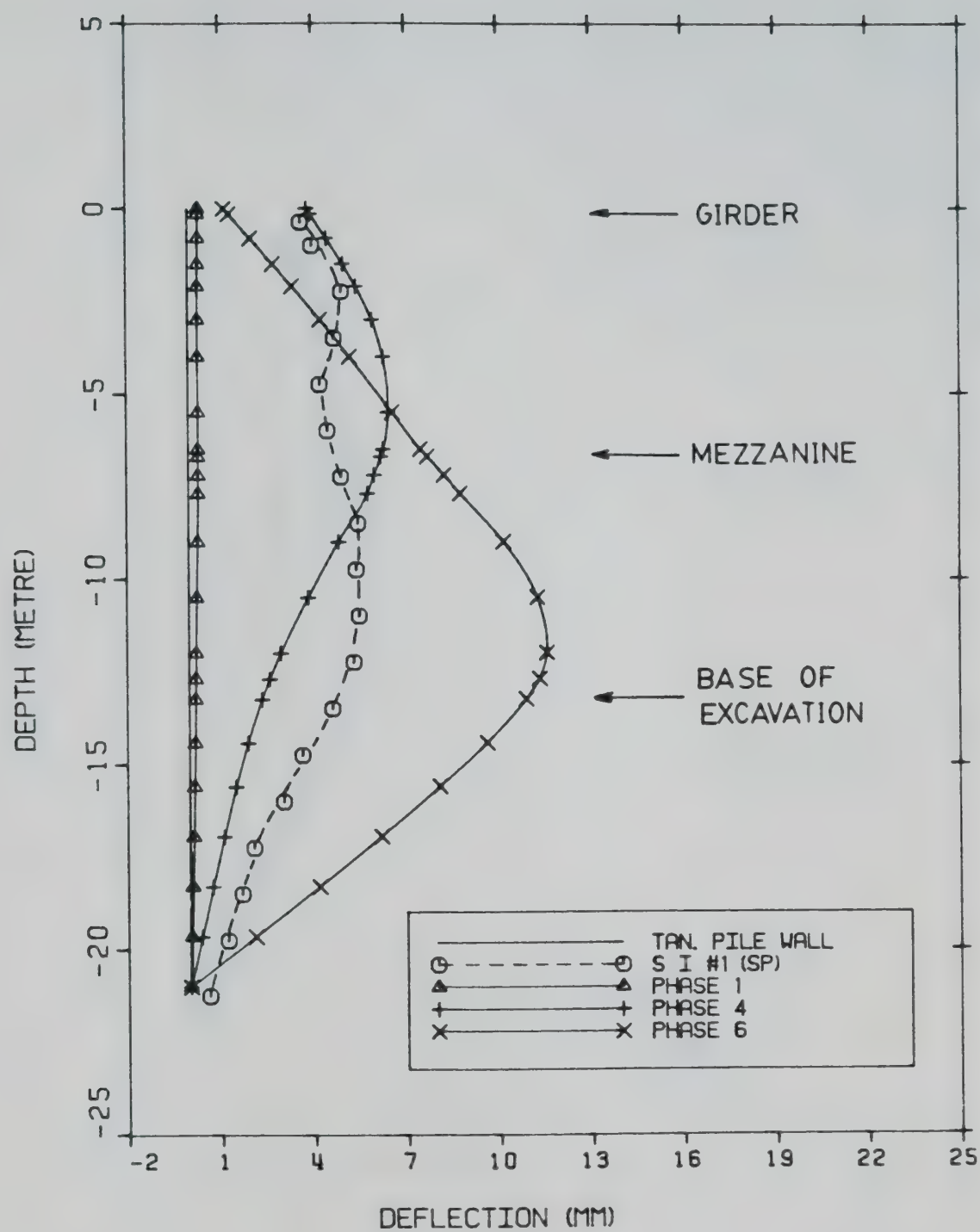


Figure 7.7 Computed deflections and field measurements of 107 Street Station, stress path

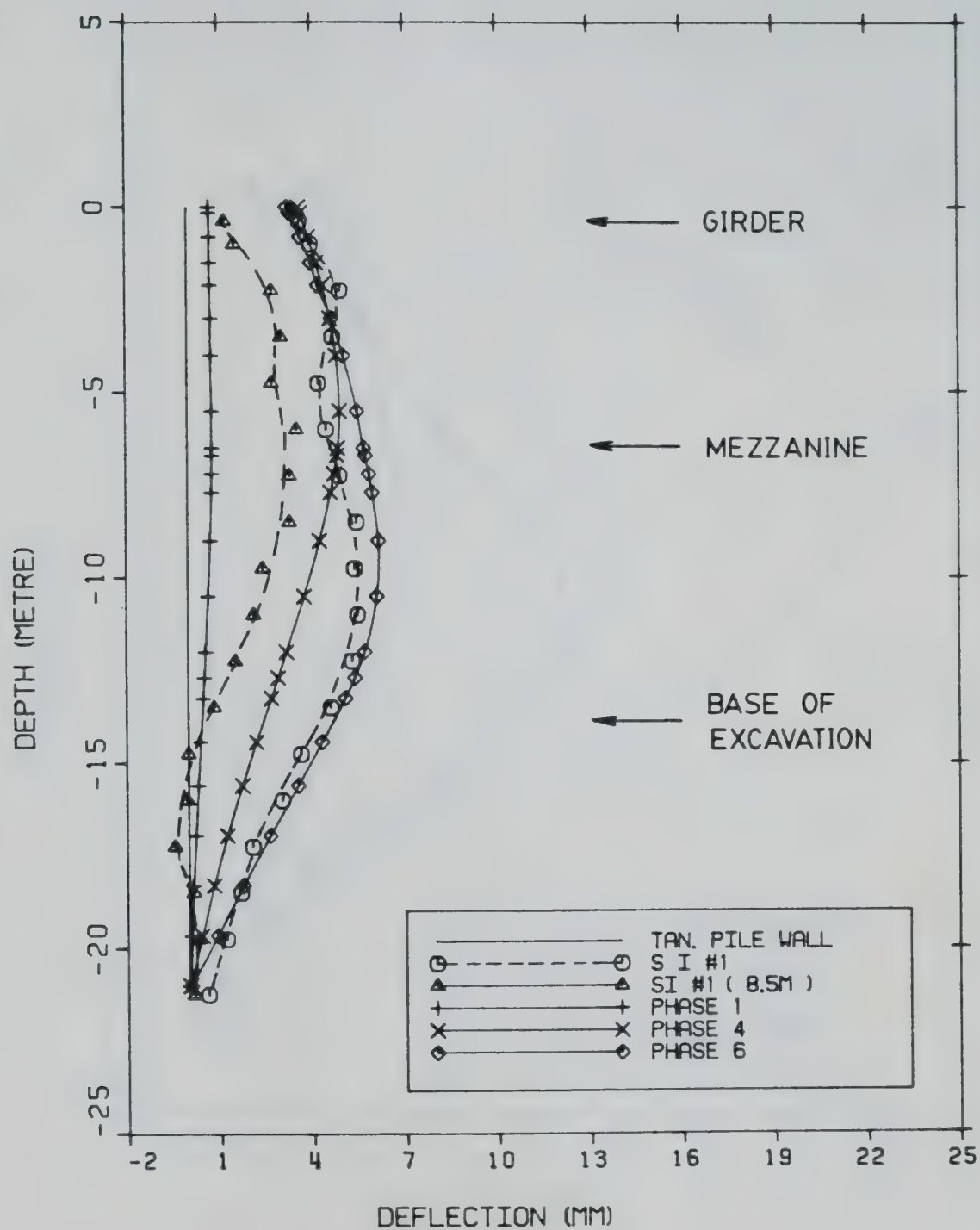


Figure 7.8 Computed deflections and field measurements of 107 Street Station, elastic

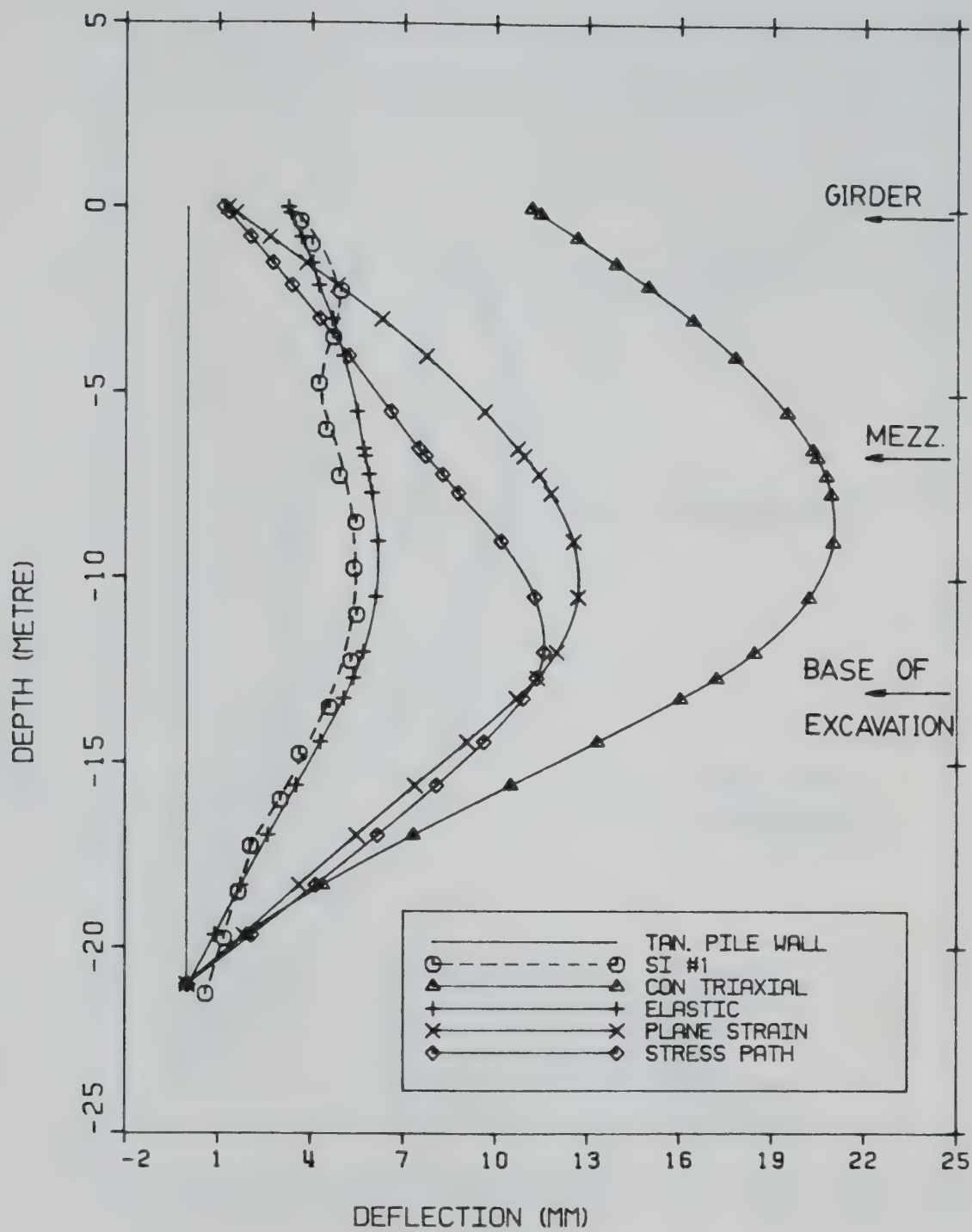


Figure 7.9 Computed deflections and field measurements of 107 Street Station

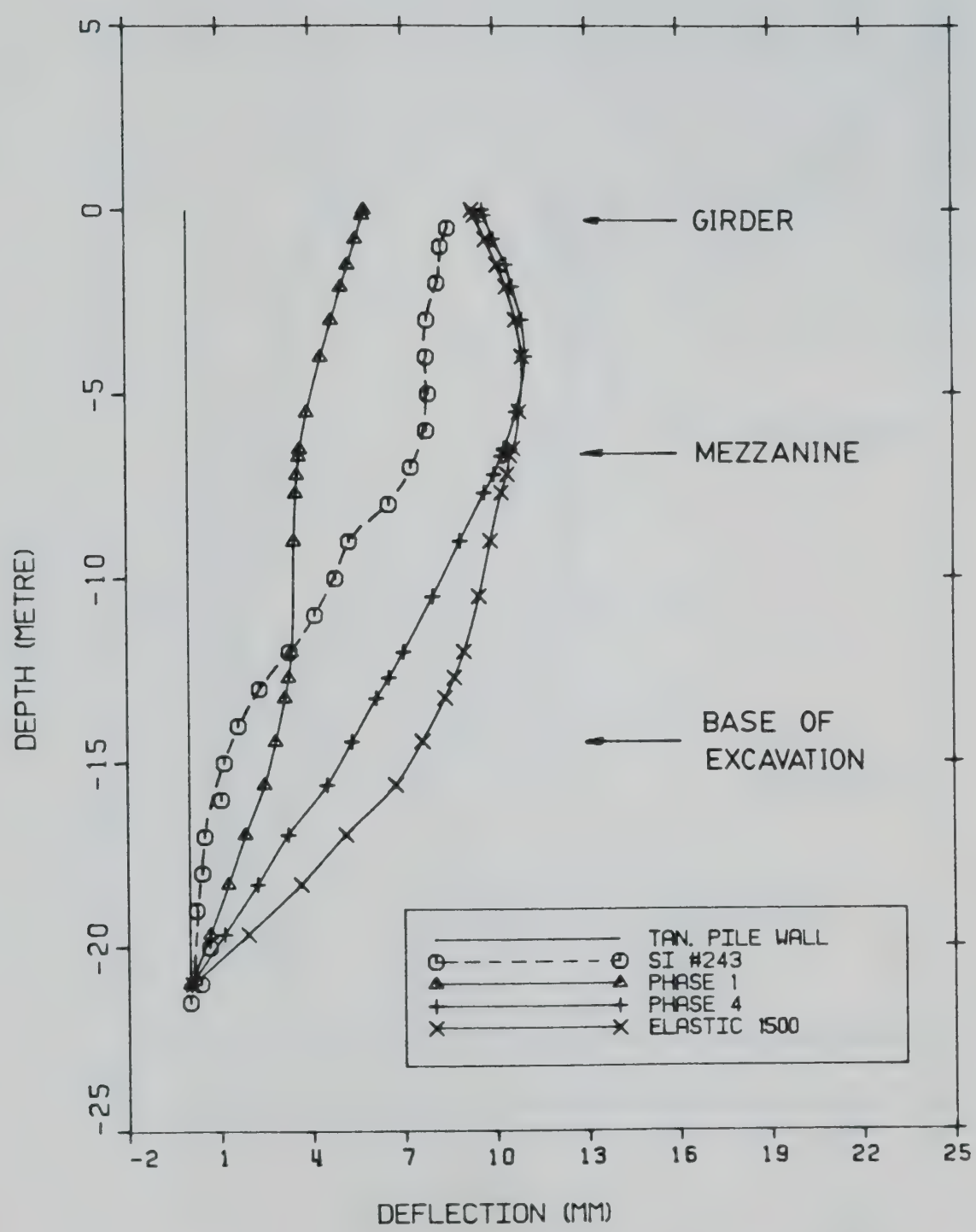


Figure 7.10 Horizontal deflection of 104 Street Station

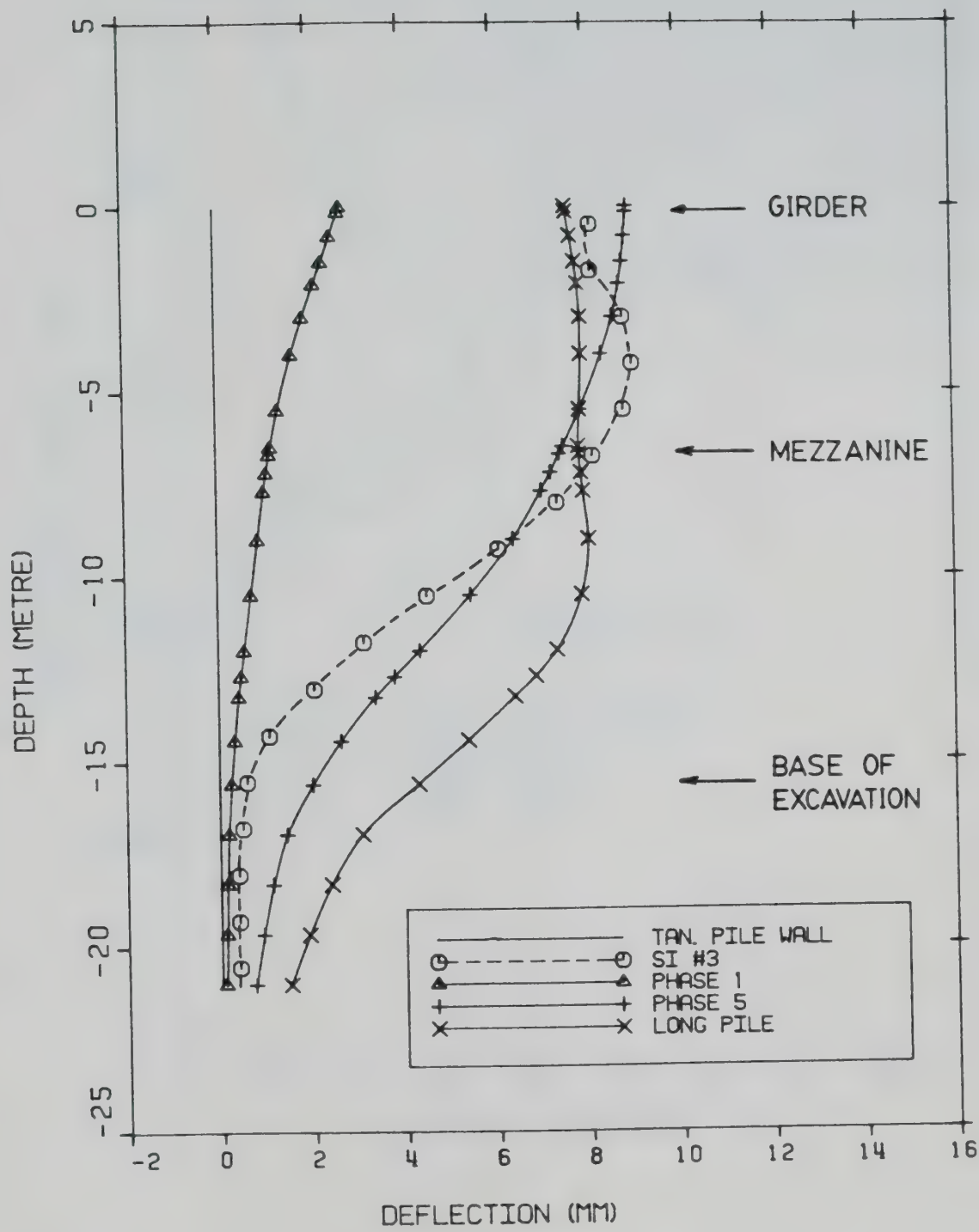


Figure 7.11 Horizontal deflection of Churchill Station

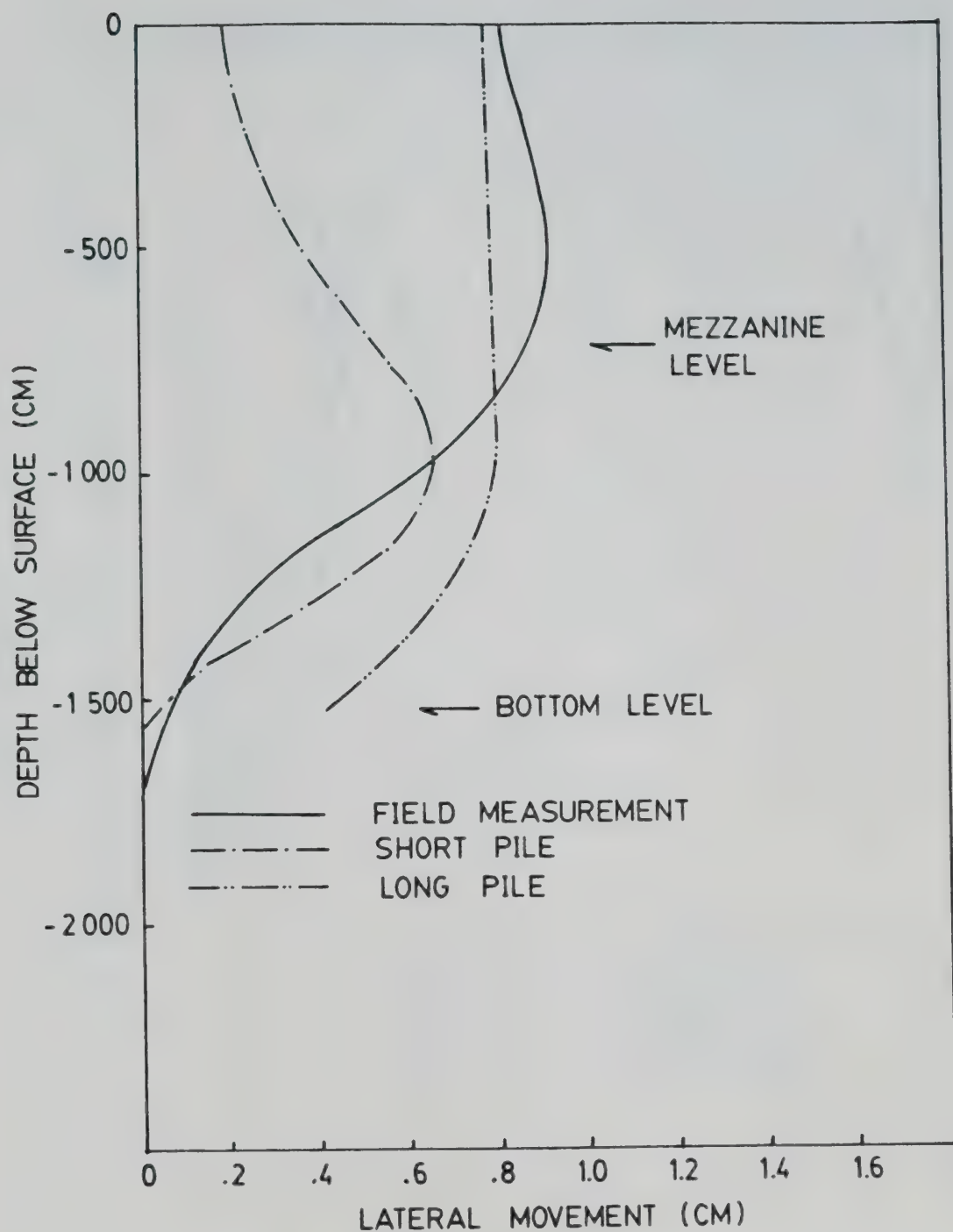


Figure 7.12 Deflection of long and short pile in Churchill Station

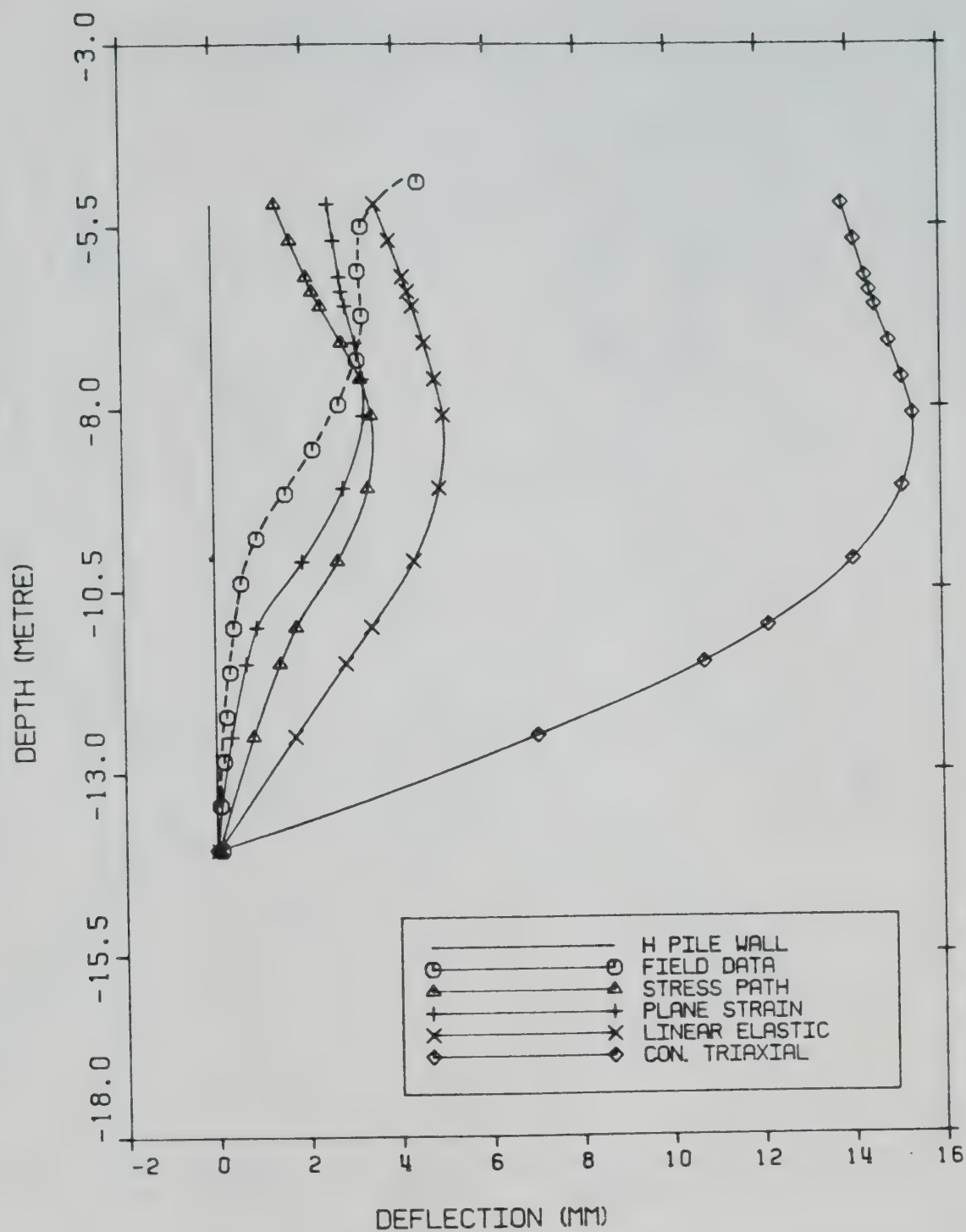


Figure 7.13 Computed deflections and field measurements of 106 Street and 100 Avenue

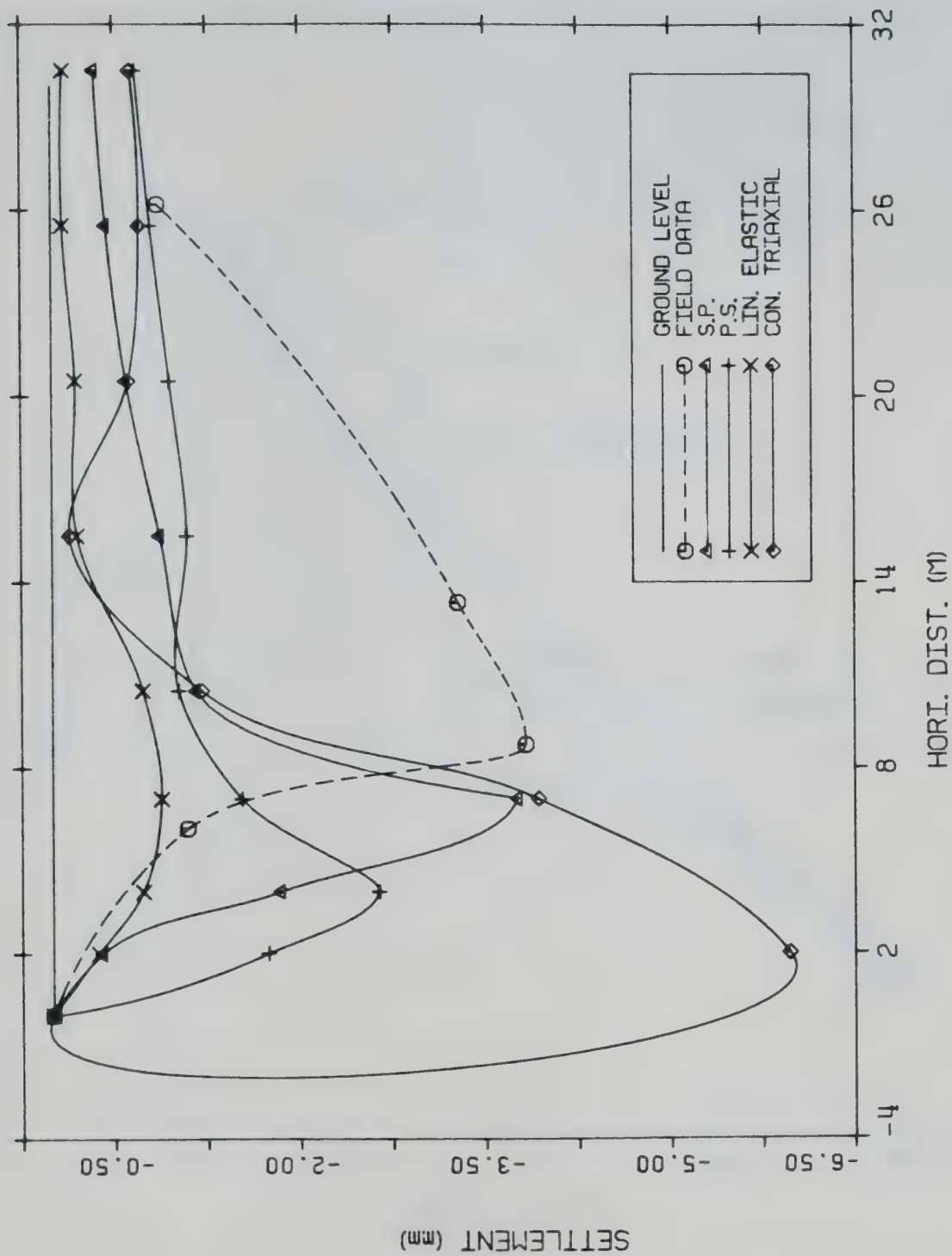


Figure 7.14 Vertical displacements of 107 Street Station

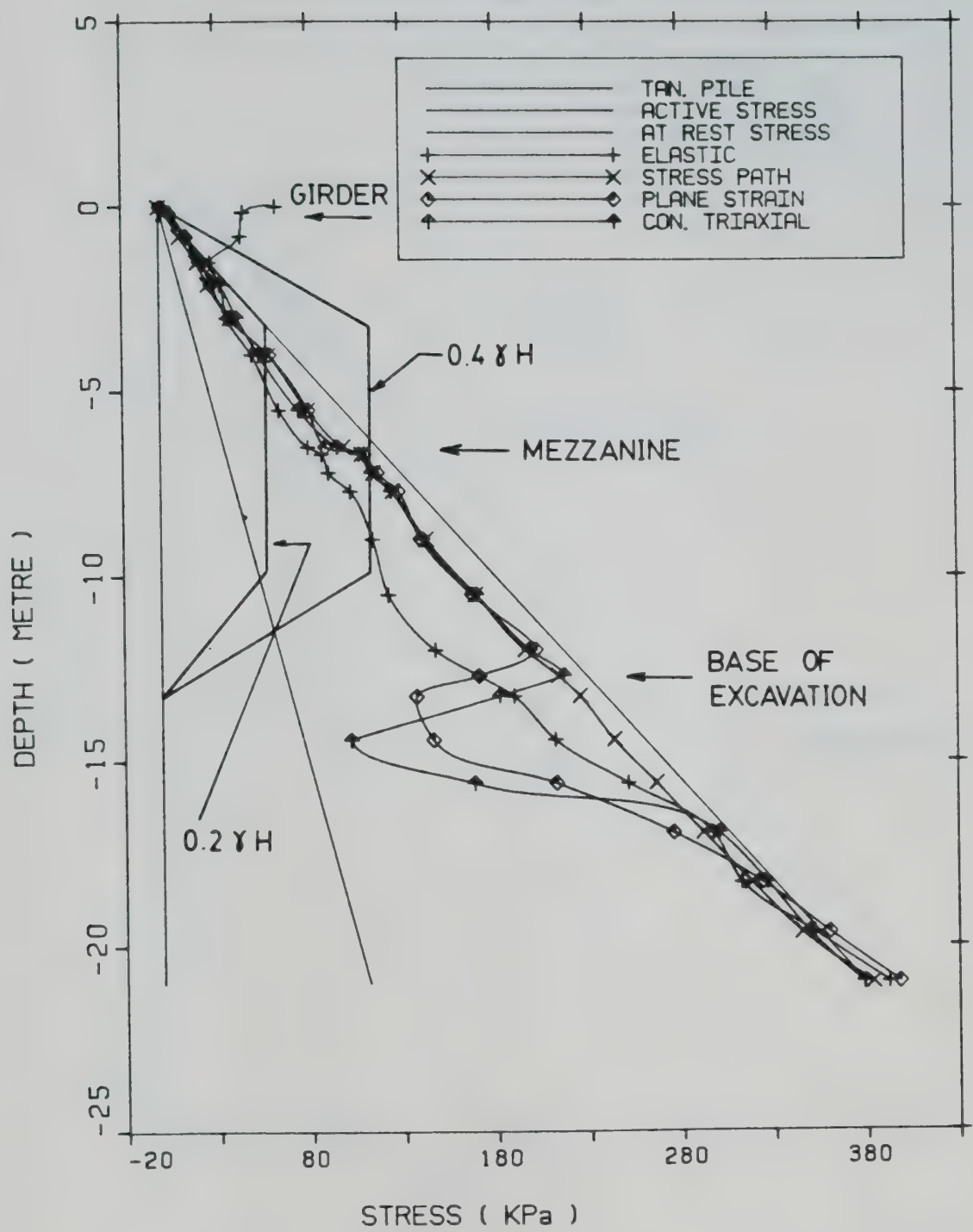


Figure 7.15 Lateral stress distribution of 107 Street Station

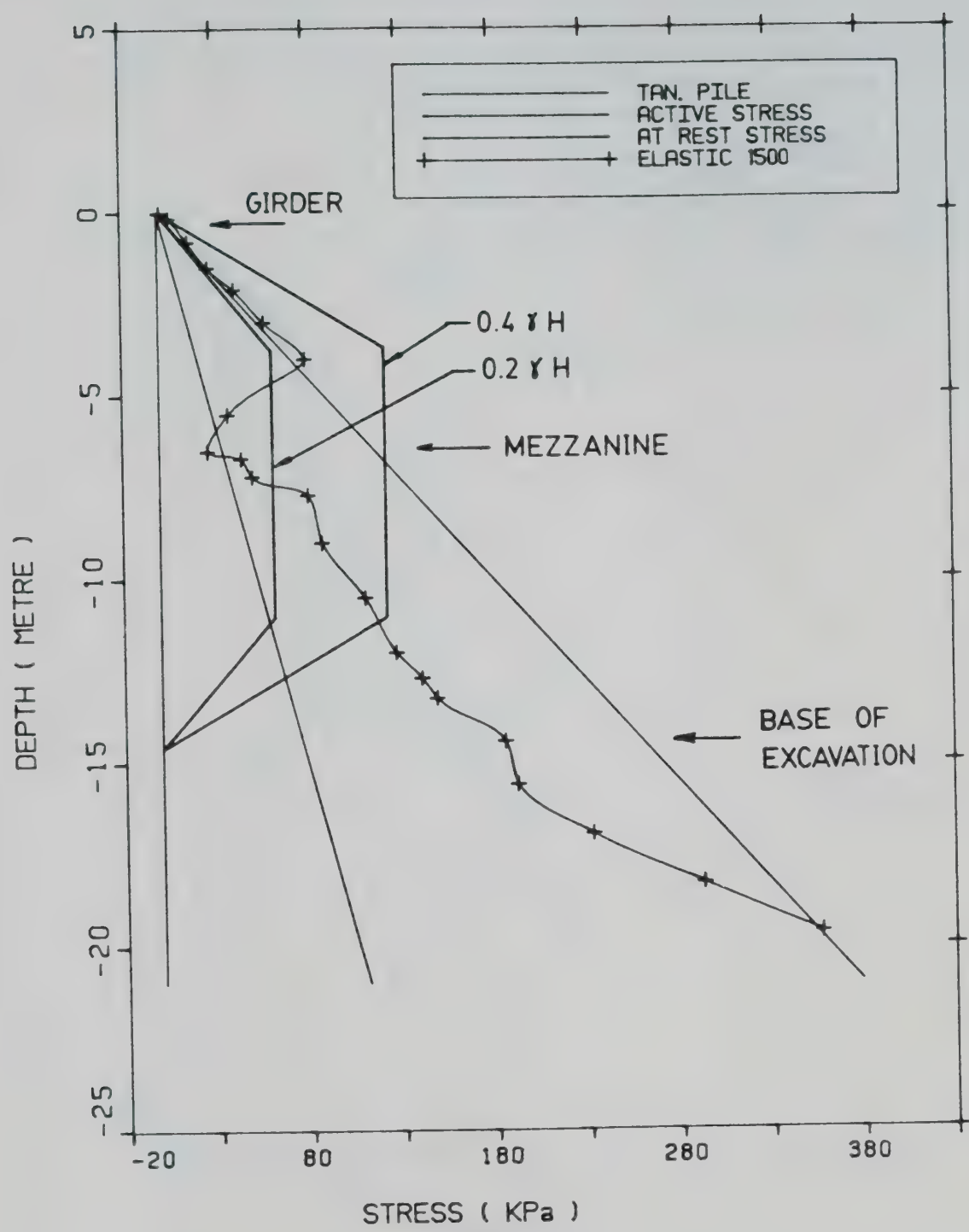


Figure 7.16 Lateral stress distribution of 104 Street Station

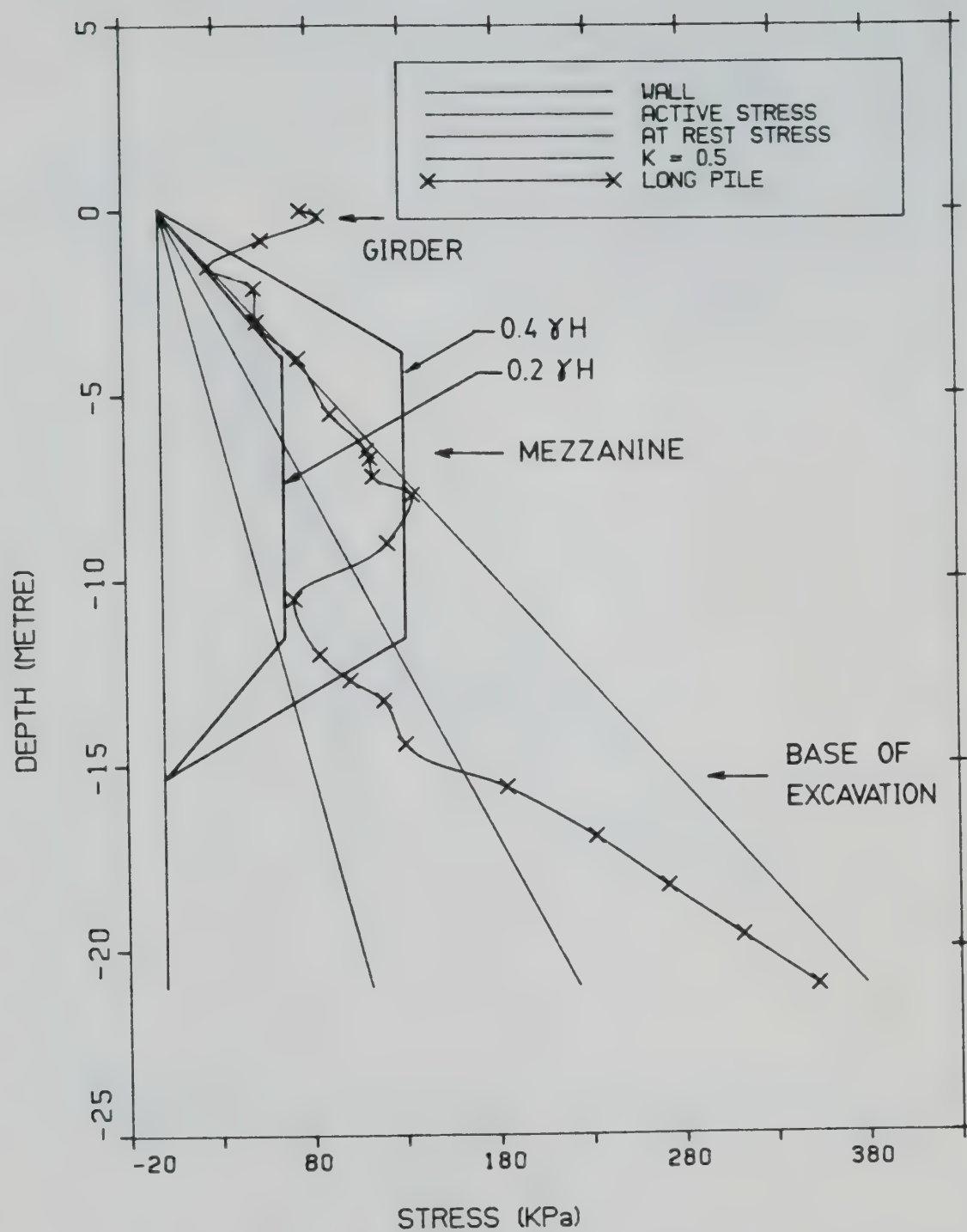


Figure 7.17 Lateral stress distribution of Churchill Station

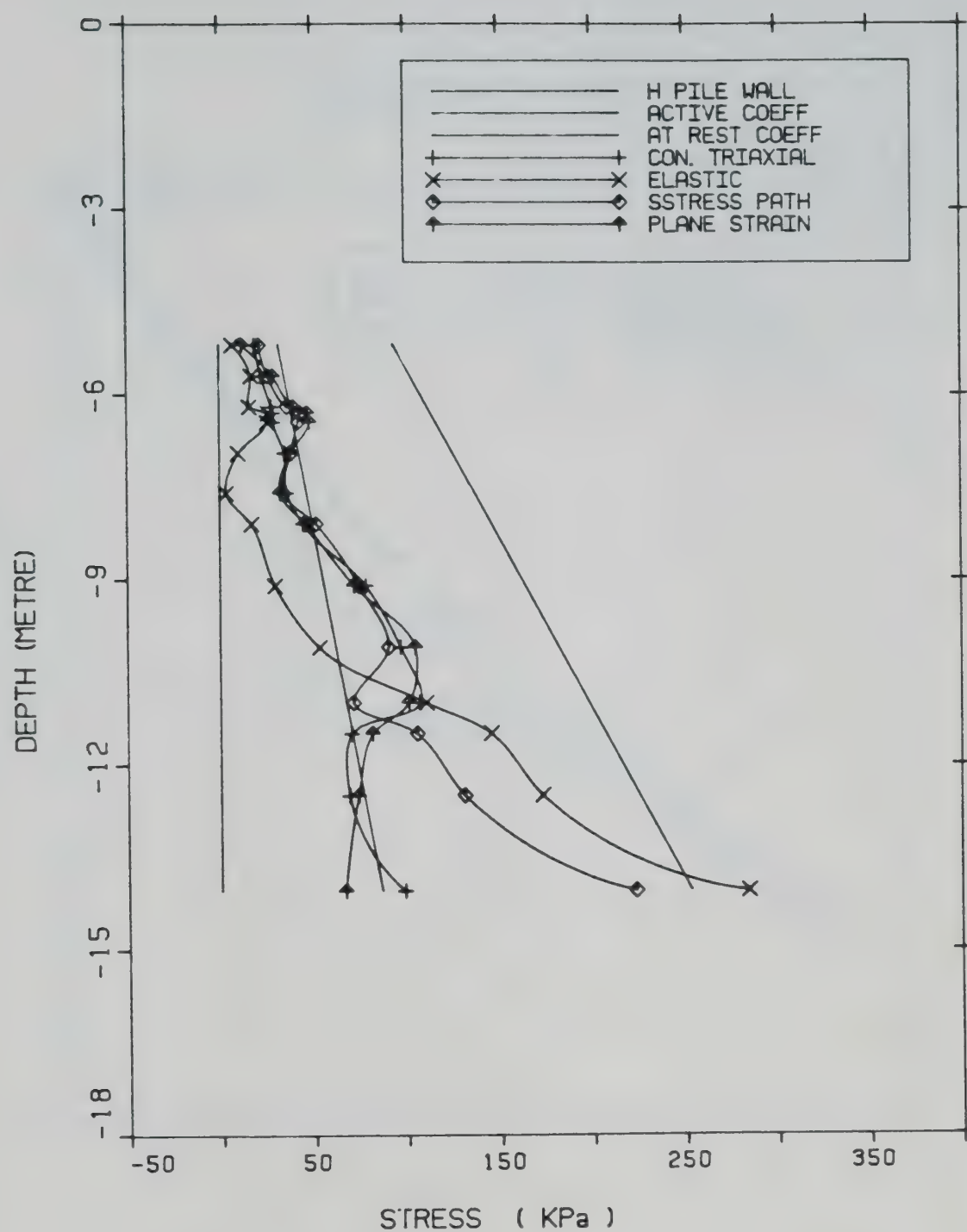


Figure 7.18 Lateral stress distribution of 106 Street and 100 Avenue

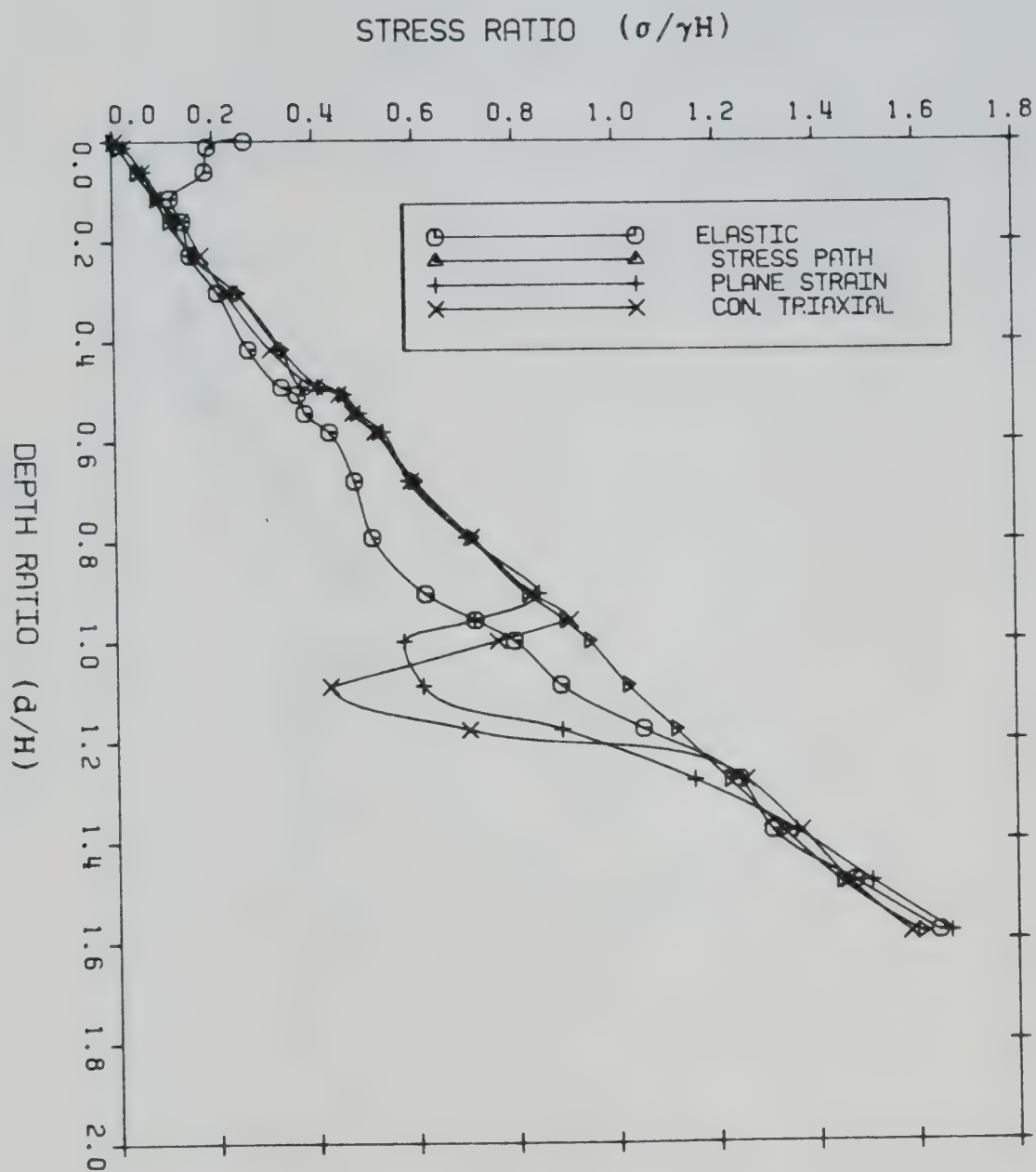


Figure 7.19 Stress ratio verses depth ratio, 107 Street Station

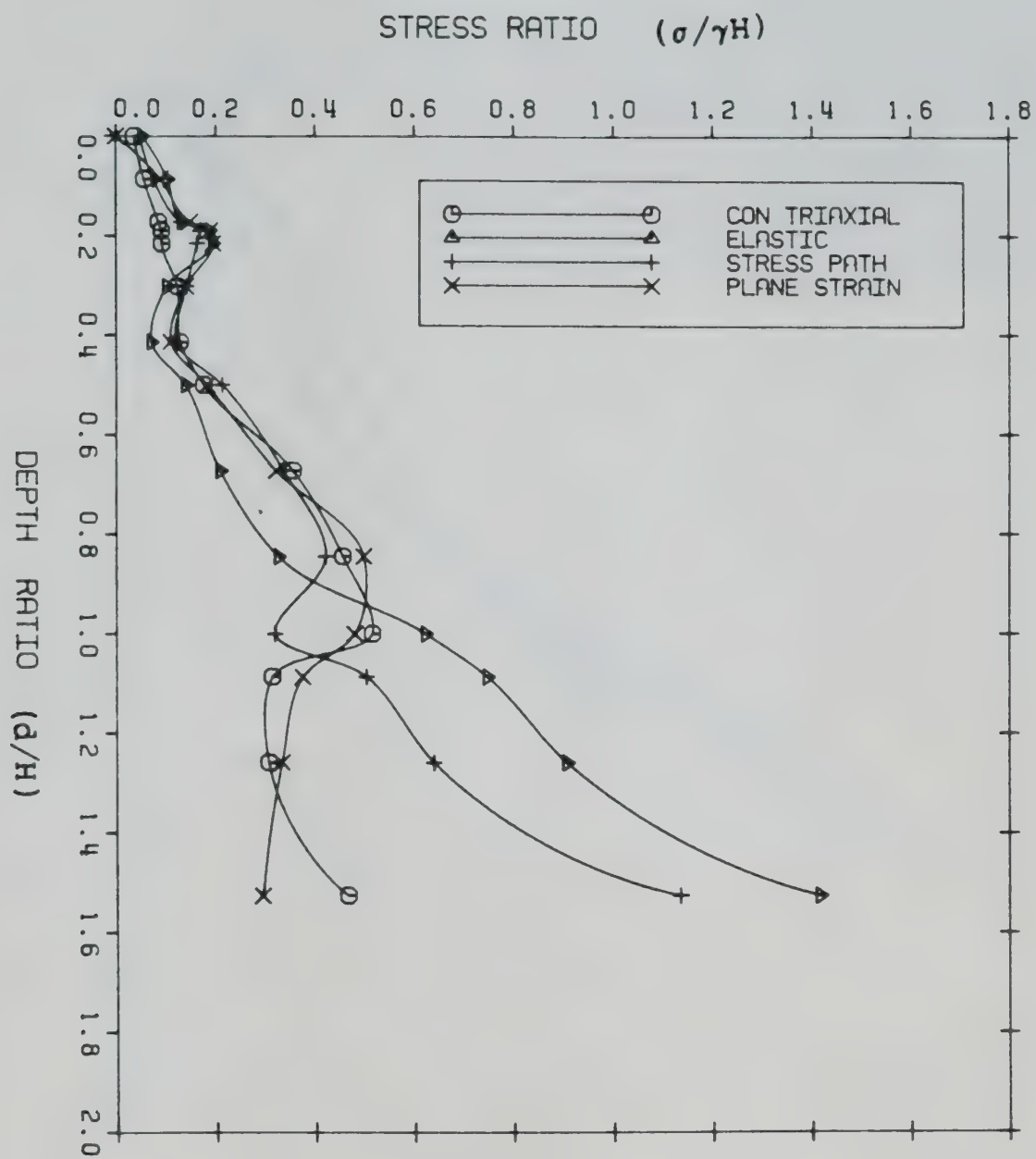


Figure 7.20 Stress ratio verses depth ratio, 106 Street and 100 Avenue

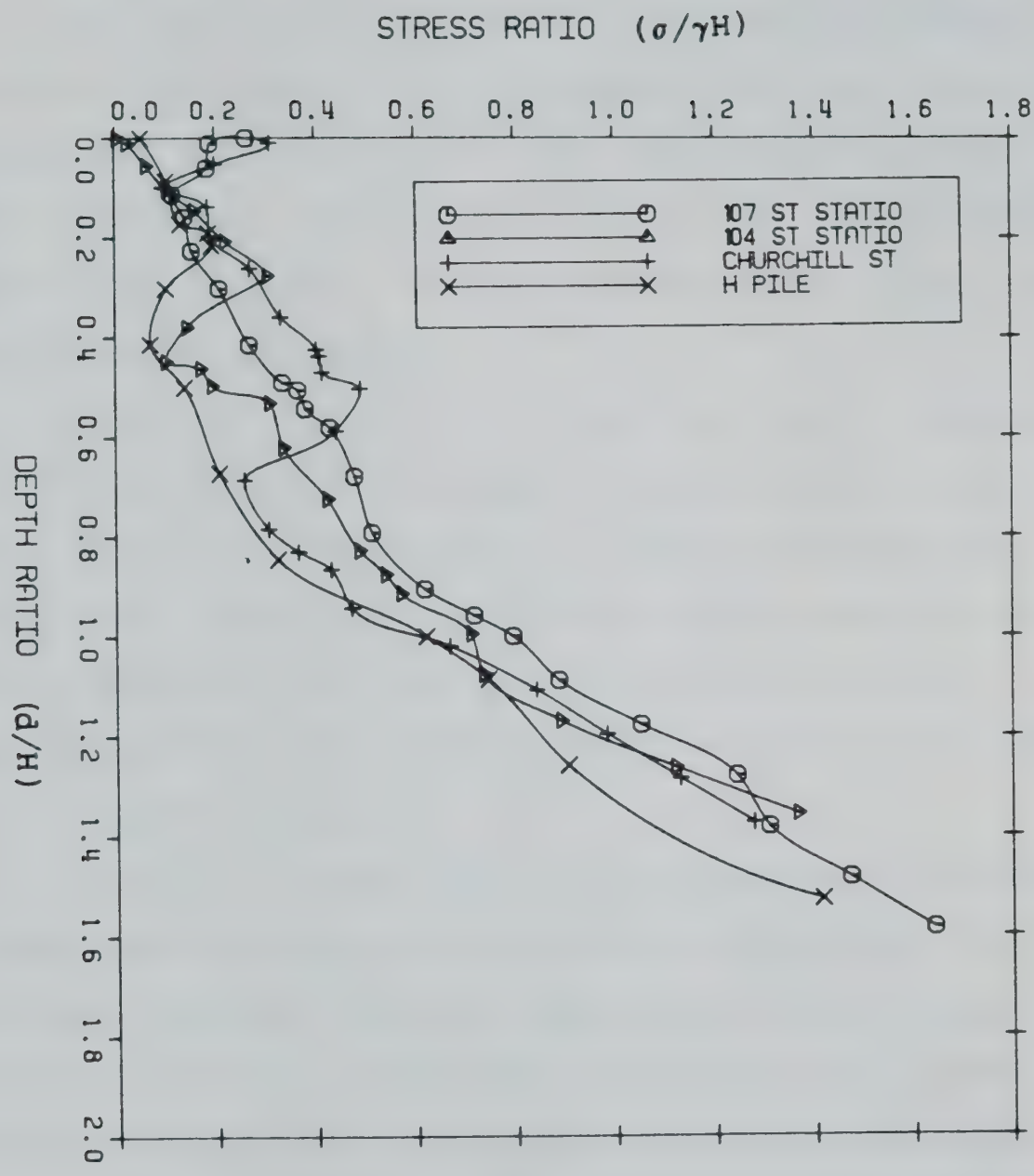


Figure 7.21 Comparison of the 3 tangent pile and sheet pile wall

8.0 Conclusion and recommendation

8.1 Conclusion

From the results presented in the previous chapter the wall stiffness had little influence on the horizontal deflection. The stiffness of the tangent pile wall for the LRT stations was approximately sixty times the stiffness of the soldier pile wall, but the deflection at various depth ratio were in agreement.

On the other hand, the stress distribution computed for the soldier pile wall was smaller than from the tangent pile wall. The agreement between the computed and measured deflections support the calculated lateral pressure as the actual pressure along the wall since the calculated deflections are more sensitive to material behaviour than the calculated stresses.

At the 107 Street Station, the computed deflections showed good agreement with the field measurements. The calculated lateral pressure compared to the semi-empirical lateral pressure diagram from Terzaghi and Peck (1967) are shown in figure 8.1. The pressure diagram recommended by Terzaghi and Peck is conservative for the upper portion of the wall but underestimated the pressure at the base of the wall. The same phenomenon was observed for the soldier pile wall and is illustrated in figure 8.2. This excess in apparent design pressure recommended by Terzaghi and Peck can result in a conservative and potentially uneconomic

design of braced and tie back walls.

A design pressure diagram which reflects the field conditions more closely is suggested in figure 8.1 and 8.2. The pressure increases linearly to the base of the excavation with a magnitude of $0.25\gamma H$ and $0.6\gamma H$ for the soldier pile and the tangent pile respectively. In figure 8.2, the recommended pressure diagram for the soldier pile gives a total force of 960 kN for which the maximum prestress load recorded was 425kN. The recommended total force is 2 times greater than the measured load. On the other hand, figure 8.1 suggests another pressure diagram for the semi-rigid retaining wall. A larger magnitude of stress distribution is suggested due to the high rigidity of the wall.

The stiffness of the strut did not affect the shape of deflection as observed in field measurements but reduced the deflection of the whole wall. For the tie back in the soldier pile wall, the same phenomenon was observed. From those case studied in this project, the stiffness of the wall did not affect the deflection significantly as the soil remained elastic since the deformations were small. If the soil was allowed to move sufficiently, the elastic assumption would not be valid. From the field measurements, the deflection to depth ratio for the 107 Street Station and the soldier pile wall were 0.00046 and 0.00041 respectively.

At the 104 Street Station, the computed deflection did not give a close match to the field measurements as in the

analysis of 107 Street Station. The difference between these two analyses was excavation of the tunnel before construction of the 104 Street Station.

For the Churchill Station, the analysis of the long pile did not give as successful a prediction of the horizontal displacements as with the short pile analysis. The results of the analysis showed a good prediction above the mezzanine floor but overestimated the deflection at the base of the excavation due to the restriction of the soil movement beneath the excavation.

8.2 Recommendation

The deflection at the 107 Street Station showed good agreement with the field measurement and the analysis based on linear elastic assumption. The vertical settlement predicted for a linear elastic material underestimated the field measurements. This was due to a possible anisotropy soil behaviour. Although the effect of anisotropy on the modulus of deformation was not significant in London clay (Costa-Filho, 1984), it is suggested to get the modulus of deformation in the vertical direction to simulate the actual field conditions within Edmonton. Also there is no field measurements for the modulus of deformation in the vertical direction made in Edmonton till. Besides, the computer program should be modified to allow the use of the two modulus of deformation.

In the analysis of the soldier pile wall, only the soldier pile section was analysed since instrumentation on the timber lagging was not carried out. For complete analysis, slope indicator should be installed at the back of the timber lagging. Due to the supporting condition of the timber lagging, it can be an analog to a one way slab supported by steel beams. Therefore this analysis should be performed for the horizontal direction. For a better understanding of the performance of a soldier pile timber lagging wall, three dimensional analysis may give a better prediction.

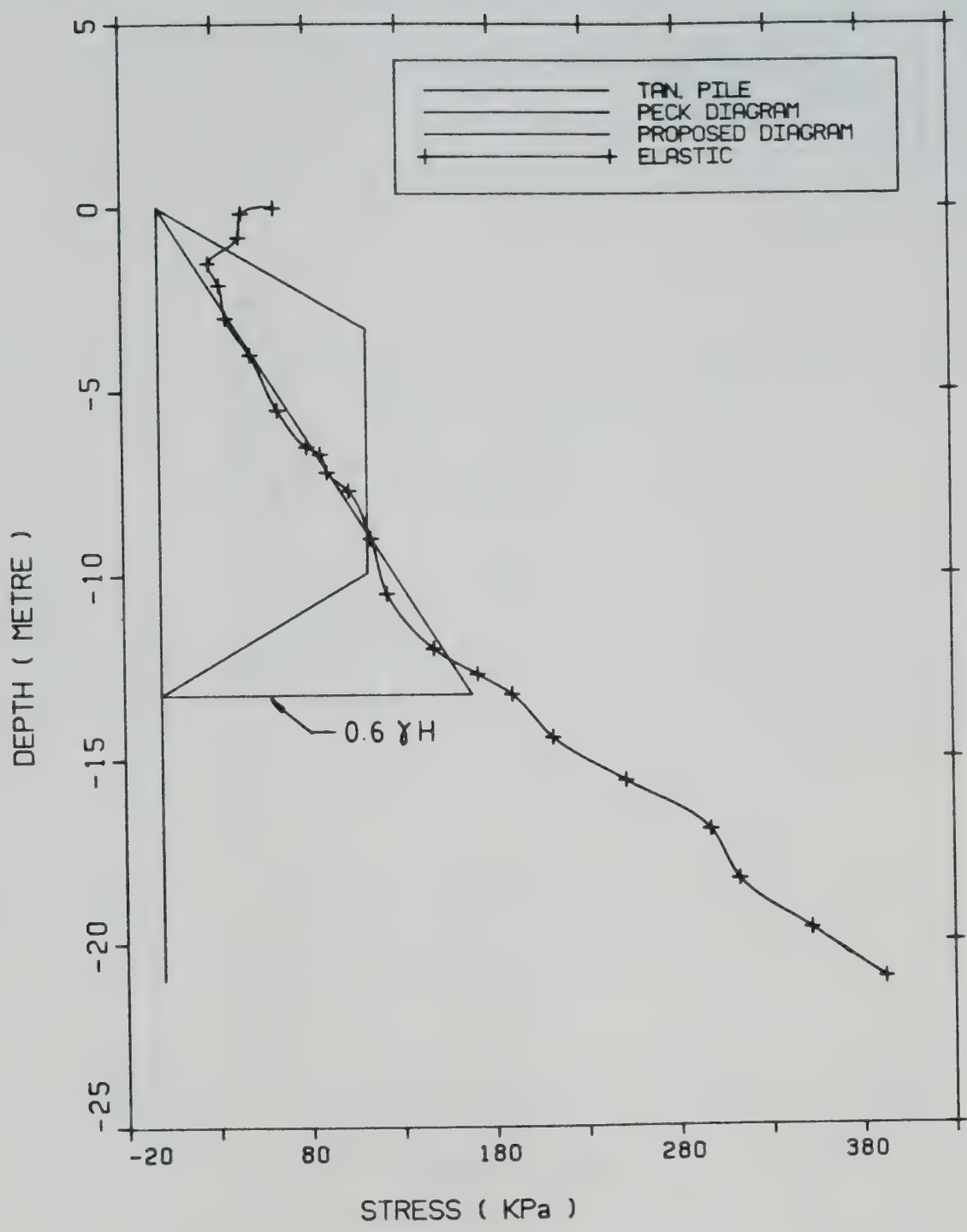


Figure 8.1 Proposed stress distribution for the tangent pile wall

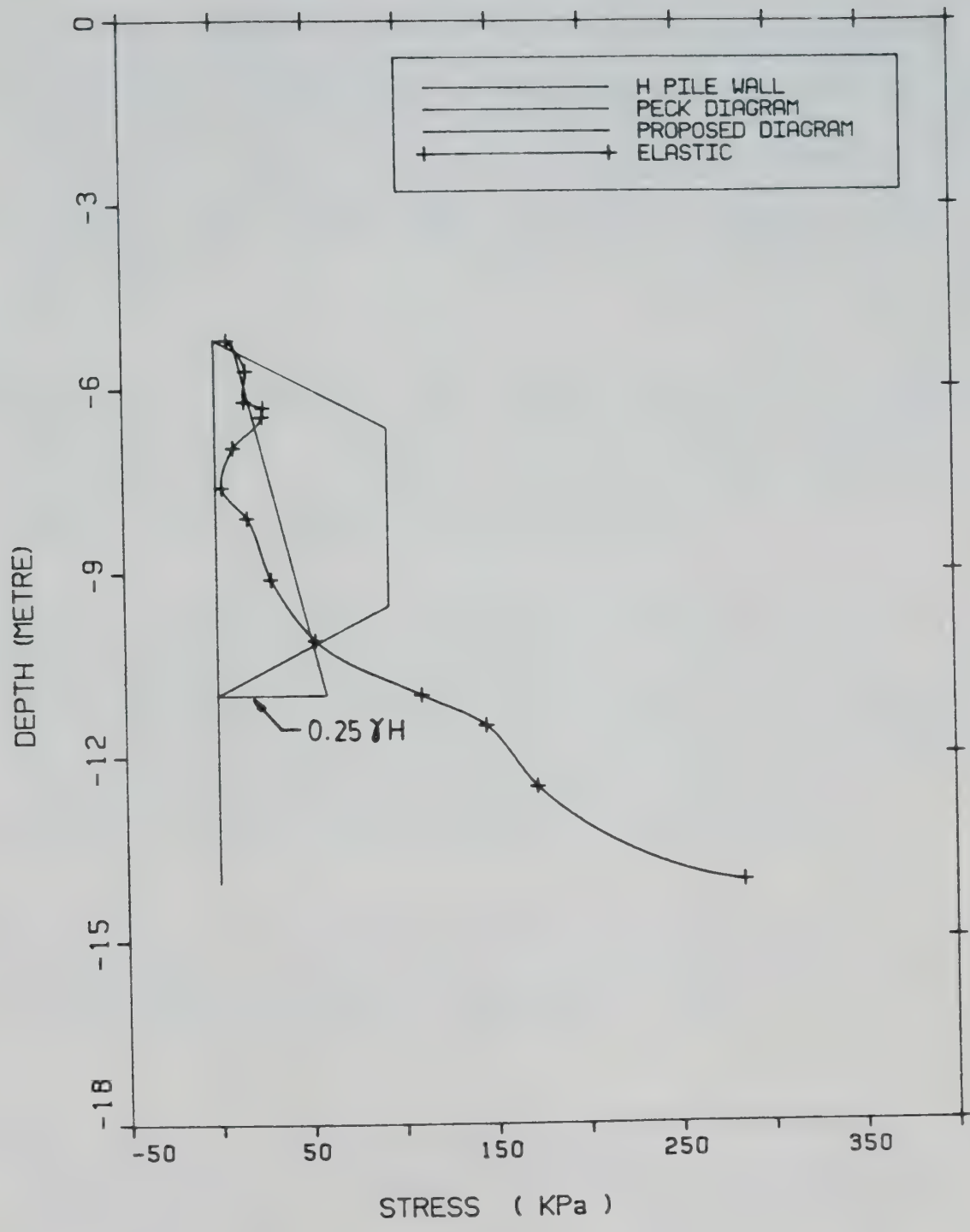


Figure 8.2 Proposed stress distribution for the soldier pile wall

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Appendix A

Computer Program

Introduction

This computer program was developed by Medeiros (1979). Only constant strain triangle elements are used to analysis stress changes and displacements caused by excavations in soil or rock.

The program allows layer excavation to a maximum of 7 phases and installation of permanent struts at each layer. In addition, some new functions are added to the program to allow for changes in boundary conditions, installation of prestressed anchors and change of material properties due to excavation. Each phase is subdivided in increments to allow updating of the elastic parameters in each element. The program at the beginning of each phase calculates the new load vector which will be applied at the nodes. It performs the calculations for different types of stress strain relationships according to the stress path which has been prescribed for the material in the element.

Organization of the program

This section is quoted from Medeiros (1979).

The main program obtains the initial nodal and element stress prior to the excavation by calling the subroutine NLOAD the element excavated are assigned extremely low values of the modulus of deformation. The elements representing the strut level to be installed are assigned

the correspondent values of the elastic parameters (concrete or steel) and the nodal loads caused by the excavation in this phase are determined. At this point the load is divided in 4 increments. The number of increments can be easily changed by altering lines 81 to 84 if required by the user. The subroutine DETE is then called to calculate the value of the modulus of deformation. The program has the capability of analysing 6 different types of stress strain relationship which are described in detail in Chapter 6. A subroutine CST is called to perform the finite element analysis using constant strain triangles to obtain the stress change due to the increment just performed. An option to plot the displacement is available at the end of the program.

Input data

1. Geometry

a. Output options card

IPRINT, NIPRINT, IPLOT and NIPRIE (4I5)

- 1) IPRINT - if equals to 1 it prints all the geometric information at the beginning of each phase for every increment.
- 2) NIPRINT - if equals to 1 it prints the nodal load for each phase of excavation.
- 3) IPLOT - if equals to 1 it generates a plot with the displacements of some selected nodes.

- 4) NIPRIE - if equals to 1 it prints the modulus of deformation of all the elements at the beginning of each phase for every increment.

b. Title (20A4)

It allows the input of the description of the analysis.

c. Size of the problem

NUMNP, NUMEL and NUMAT (3I6)

- 1) NUMNP - number of nodal points
- 2) NUMEL - number of elements
- 3) NUMAT - number of materials

d. node cards

Number of nodes equal to number of cards. Each card containing N, KNODE(N), X, Y, U, V and Kode1(N)
(2I6,4F12.0,I5)

- 1) N - node number
- 2) KODE(N) - two digit number indicating whether a displacement or a force is applied at the node where
 - (a) 00 - x force y force
 - (b) 11 - x displacement y displacement
 - (c) 10 - x displacement y force
 - (d) 01 - x force y displacement
- 3) X - x coordinate
- 4) Y - y coordinate
- 5) U - x force or displacement at node N
- 6) V - y force or displacement at node N

- 7) Kode1(N) - it defines the material type to determine initial nodal stress. If KODE1(N)=4 the initial stresses are zero.

Note. Nodal points must be in numerical order. If nodes are omitted the program generates coordinates for the intermediate points linearly displaced between the two points. For the generated nodes the program assigns KODE=0, U=0 and V=0 and KODE1 equals to the previous node.

e. element cards

Number of elements equal to number of cards.
Each card containing M, NP(I,M), MAT(M), and TH(M)
(5I6,F10.6)

- 1) M - element number
- 2) NP(I,M) - nodal points at the three corners of the element in counterclockwise order
- 3) MAT(M) - element material number
- 4) TH(M) - thickness of the element. If not defined the program assigns a unit thickness.

Note. Element cards must be in numerical order. If element numbers are omitted the program generates element data in modules of two elements from the previous two.

2. Plotting data

- a. General information, NNTP, NPH and NMATC (3I5)
 - 1) NNTP - number of nodes to be plotted
 - 2) NPH - total number of phases
 - 3) NMATC - phase number that the displacements are resetted to zero
- b. Nodes (10I5)

NTP(I) - node number which displacements will be plotted. 10 node numbers can be inputted in each card.
- c. Title of the plotting (20A4)

TITLE(I)

 - 1) column 1 to 48, headline for the plotting
 - 2) column 49 to 64, abscissa label
 - 3) column 65 to 80, ordinate label
- d. Contour (I5)

NNG - number of nodes which wil form the contour to define the excavation, retaining structure, boundaries and subsoil. A line wil be drawn joining all the NNG points without interruption.
- e. Contour definition (10I5)

NTP(I) - nodes defining the contour. If it is not required to use the plotting facility 5 blank lines have to be supplied to inform nothing is to be plotted. 10 nodes can be inputted in one card.

3. Material properties

This program was developed for 10 different materials to suit the needs of various analyses of the Rapid Transit in Edmonton. The nonlinear elastic stress strain relationship is fitted to a hyperbolae:

$$\sigma_1 - \sigma_3 = \epsilon_1 / (a + b\epsilon_1)$$

where

σ_1, σ_3 = major and minor principal stresses

ϵ_1 = major principal strain

a, b = material parameters required

a. Material number 1

This material corresponds to a stress path of the type compression active in convention triaxial equipment. Three stress strain curves are to be supplied.

1) MM - material number 1 (15)

2) (a) PR(1) - Poisson ratio

(b) KO(1) - at rest earth pressure coefficient
(2F10.5)

3) Three lines, each one containing:

(a) SIGC(1,J) initial confining pressure

(b) AA(1,J) Hyperbolae parameter

(c) BB(1,J) Hyperbolae parameter (3F10.6)

b. Material number 2

This material corresponds to a stress path of the type extension active in conventional triaxial equipment. Three stress strain curves are to be supplied.

- 1) MM - material number 2 (I5)
- 2) (a) PR(2) - Poisson ratio
(b) KO(2) - at rest earth pressure coefficient (2F10.5)
- 3) Three lines, each one containing:
 - (a) SIGC(2,J) confining stress
 - (b) AA(2,J) Hyperbolae parameter
 - (c) BB(2,J) Hyperbolae parameter (3F10.6)

c. Material number 3

This material corresponds to a stress path of proportional loading-compression active test in triaxial equipment. One stress strain curve is to be supplied. For the proportional loading part linear elasticity is used and for the compression active part a hyperbolic relationship.

- 1) MM - material number 3 (I5)
- 2) (a) E(3) - modulus of deformation for the proportional loading
(b) AA(3,1) Hyperbolae parameter
(c) BB(3,1) Hyperbolae definition (3F10.6)

d. Material number 4

This material behaves linearly elastic. It was used for the retaining structure.

- 1) MM - material number 4 (I5)
- 2) (a) PR(4) - Poisson ratio
- (b) KO(4) - at rest earth pressure coefficient (2F10.5)
- (c) E(4) - modulus of deformation (F10.6)

e. Material number 5

This material corresponds to a stress path of the type compression active in plane strain apparatus. Two stress strain curves are to be supplied.

- 1) MM - material number 5 (I5)
- 2) (a) PR(5) - Poisson ratio
- (b) KO(5) - at rest earth pressure coefficient (2F10.5)
- 3) Two lines each one containing
 - (a) SIGC(5,J) initial confining stress
 - (b) AA(5,J) Hyperbolae parameter
 - (c) BB(5,J) Hyperbolae parameter (3F10.6)

f. Material number 6

Similar to material number 4.

g. Material number 7

This material corresponds to a stress path of the type compression passive in conventional triaxial equipment. Three stress strain curves are to be supplied.

- 1) MM - material number 7 (I5)
- 2) (a) PR(7) - Poisson ratio

(b) KO(7) - at rest earth pressure coefficient

(2F10.5)

3) Three lines each one containing:

(a) SIGC(7,J) confining stress

(b) AA(7,J) Hyperbolae definition

(c) BB(7,J) Hyperbolae definition (3F10.6)

h. Material number 8

Similar to material number 4. It was used to define the properties of the material after it has been excavated.

i. Material number 9

Similar to material number 7 with only one stress strain curve definition.

j. Material number 10

Similar to material number 4. It was used to define the different material which was used for grouting the struts and the vertical wall.

In material numbers 1, 2, 3, 5, 7 and 9, the AA and BB are parameters for fitting a hyperbolae to the appropriate type of test. The paper "Non linear analysis of stress and strain in soils" (Duncan and Chang, 1970) gives detail explanation of the parameters.

4. Initial stress

These data will be used by subroutine INIT. If the material definition is of the type 4 the initial element stress is set to zero.

- a. Gama specific weight
- b. ZSFC ordinate of the ground surface (2F10.3)

5. Description of the phases of construction

This set of data is required by subroutine NLOAD. The elements which will be excavated and the elements which will be loaded have to be concreted or grouted and the nodes which will be loaded have to be specified. The nodal load is determined from the elements to be excavated adjacent to the node.

a. Phase description

NPHASE, NYL and NXL (3I5)

- 1) NPHASE - phase number, if zero it indicates the final phase was defined in the previous call to NLOAD.
- 2) NYL - number of nodes vertically loaded.
- 3) NXL - number of nodes horizontally loaded.

b. Nodal load definition

- 1) vertical load definition NYL sets of two lines
 - NELI, NNYL(I) (2I5)
 - IEL(J) (10I5)
 - (a) NELI - number of elements excavated adjacent to this node

- (b) NNYL(I) - node number
- (c) IEL(J), J=1, NELI elements excavated number,
adjacent to NNYL(I)
- 2) horizontal load definition NXL sets of two lines
similar to the vertical load definition.

6. Input of prestress anchor loads

- a. NNC - number of nodes loaded (I5)
- b. J, U(J) and V(J) - node number and prestress loads
in horizontal and vertical direction (I5, 2F10.2)

7. Load tranfer

The program allows for the transfer of horizontal loads from a node to two others by beam effect. This feature accomodate the existance of sheet piles transferring their loads to points of contact with the structure when the deformation of the wall is required.

- a. NLD - number of nodes which will transfer the
horizontal. If NLD equals to zero no more load
transfer input is required. (I5)
- b. (a) NA - upper node carrying the transferred load.
(b) NB - lower node (2I5)
- c. NU - number of the node which will have its load
transferred to NA and NB. NLD lines to be supplied.

(I5)

8. Element material redefinition

a. Element excavated

(i)NELCA - number of elements excavated (I5)

(ii)INEL - element excavated. After this phase this element will be of the type number 8. Number of element excavated equal to number of cards input.

(I5)

b. Element changed to structural elements

(i)NELCC - number of elements changed to material number 4. After this phase this element will be of type number 4. (I5)

(ii)INEL and TH(INEL) - element number and the thickness of th of the element. NELCC lines to be supplied.

(I5)

c. Change of boundary conditions

(i) NNC - number of nodes (I5)

(ii) J - node number. NNC lines to be supplied (I5)

d. Element to be grout

(i)NELCG - number of elements changed to material number 10. If this material is under tension in any phase it will have a small value assigned to the modulus of deformation. (I5)

(ii)INEL - element number. NELCG lines to be supplied.

(I5)

e. Elements for stress path zones change due to excavation

(i) NELCC - number of elements changed. (I5)

(ii) MATLN - type of material replaced. (I5)

(iii) INEL - element number. NELCC lines to be supplied (I5)

9. nodal stress redefinition

This set of data defines the nodes which will have the stresses zeroed. They represent the nodes which are inside elements changed into concrete, grout or have been excavated.

a. NNC - number of nodes (I5)

b. J - node number. NNC lines to be supplied (I5)

Appendix B

Listing of the computer program and sample input data

```

1      COMMON E(10),PR(10),RD(10),X(400),Y(400),U(400),V(400),TH(700),
2      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
3      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
4      1 NEQ,M,LM(6)
5      COMMON KODE1(400)
6      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
7      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
8      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
9      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
10     COMMON EE(700),PRT(700),TSIGEP(700,2)
11     COMMON PHI,XK,XEP
12     COMMON DI(800)
13     DIMENSION XP(80,8),YP(80,8),NTP(80),NG(80),
14     1TITLE(20),XG(80),YG(80),XPF(10),YPF(10)
15     REAL KO
16     READ(5,200)IPRINT,NIPRIN,IPLLOT,NIPRIE
17 200   FORMAT(4I5)
18     CALL READGE
19     READ(5,450) NNTP,NPH,NMATC
20 450   FORMAT(3I5)
21     WRITE(6,451)NNTP
22 451   FORMAT(///,'          NUMBER OF NODES TO BE PLOTTED',I5,
23     1//,10X,'NODES NO')
24     READ(5,452)(NTP(I),I=1,NNTP)
25 452   FORMAT(10I5)
26     WRITE(6,452)(NTP(I),I=1,NNTP)
27     READ(5,457)(TITLE(I),I=1,20)
28 457   FORMAT(20A4)
29     READ(5,450)NNG
30     READ(5,452)(NG(I),I=1,NNG)
31     DO 456 I=1,NNG
32       NNN=NG(I)
33       XG(I)=X(NNN)
34 456   YG(I)=Y(NNN)
35     DO 453 I=1,NNTP
36       J=NTP(I)
37       XP(I,1)=X(J)
38 453   YP(I,1)=Y(J)
39     DO 80 L=1,NEQ
40 80    TB(L)=0.
41     CALL SST
42   C
43   C   CALL INITIAL STRESSES
44   C
45     CALL INIT
46     DO 59 M=1,NUMNP
47       TSIGA(M,1)=SIGIA(M,1)
48       TSIGA(M,2)=SIGIA(M,2)
49 59    TSIGA(M,3)=0.
50     DO 50 M=1,NUMEL
51       TSIGEL(M,1)=SIGIEL(M,1)
52       TSIGEL(M,2)=SIGIEL(M,2)
53 50    TSIGEL(M,3)=0.
54   C
55   C   CALL NEW LOAD
56   C
57 90    CALL NLOAD(NPHASE,NIPRIN,NPH)
58     IF(NPHASE.GT.NPH) GO TO 91
59 41    FORMAT(///,10X,19H NODAL POINT OUTPUT,///
60     11H ,59H NODE   KODE      X COORD      Y COORD      X FORCE      Y FORCE

```



```

61      2//)
62      40 FORMAT( 5X, 'OUTPUT OF COMPLETE NODAL DATA ')
63      43 FORMAT(I4,I6,F13.3,3F12.3,I5)
64      46 FORMAT(///,10X, 13H ELEMENT DATA ///,
65      1 40H ELEM      I      J      K      MAT THICKNESS //)
66      C
67      C CALCULATE PRINCIPAL STRESSES
68      C
69      DO 51 M=1,NUMEL
70      SIGM=(TSIGEL(M,1)+TSIGEL(M,2))/2.
71      SIGD2=(TSIGEL(M,1)-TSIGEL(M,2))/2.
72      RAD=SQRT(SIGD2**2+TSIGEL(M,3)**2)
73      TSIGEP(M,1)=SIGM+RAD
74      51 TSIGEP(M,2)=SIGM-RAD
75      60 FORMAT(1H1,5X,'TOTAL PRINCIPAL STRESSES',/,5X,
76      1' EL      SIGX      SIGY      SIGXY      SIG1      SIG3')
77      61 FORMAT(5X,I5,5F10.4)
78      C
79      C DIVIDE THE LOAD IN STEPS
80      C
81      DO 62 I=1,NUMNP
82      U(I)=UN(I)/4.
83      62 V(I)=VN(I)/4.
84      DO 70 IU=1,4
85      C
86      C COMPUTE E BEFORE EACH STEP
87      C
88      CALL DETE(NIPRIE,NPHASE,IU)
89      64 FORMAT(1H1,10X,11HSTEP NUMBER,I5,
90      1/,11X,16(' '),///)
91      IF(IPRINT.NE.1)GO TO 201
92      WRITE(6,64)IU
93      WRITE(6,40)
94      WRITE(6,41)
95      WRITE(6,43)(N,KODE(N),X(N),Y(N),U(N),V(N),KODE1(N),
96      1N=1,NUMNP)
97      WRITE(6,46)
98      WRITE(6,65)(M,(NP(J,M),J=1,3),MAT(M),TH(M),EE(M),M=1,NUMEL)
99      65 FORMAT(I4,4I6,F11.4,F14.3)
100     201 CONTINUE
101     C
102     C
103     C UPDATE STRESSES AND DISPLACEMENTS
104     C AFTER EACH STEP
105     C TSIGA = TOTAL NODAL STRESSES
106     C TSIGEL = TOTAL ELEMENT STRESSES
107     C TSIGEP = TOTAL PRINCIPAL STRESSES
108     C TB = TOTAL NODAL DISPLACEMENTS
109     C DI = DISPLACEMENT FOR EACH INCREMENT
110     C
111     C
112     CALL CST
113     DO 63 I=1,NUMNP
114     DO 67 J=1,3
115     67 TSIGA(I,J)=TSIGA(I,J)+DSIGA(I,J)
116     63 CONTINUE
117     DO 66 I=1,NUMEL
118     DO 68 J=1,3
119     68 TSIGEL(I,J)=TSIGEL(I,J)+DSIGEL(I,J)
120     66 CONTINUE

```



```

121      DO 69 I=1,NEQ
122      69 TB(I)=TB(I)+DI(I)
123      C
124      C   CALCULATE PRINCIPAL STRESSES
125      C
126      DO 71 M=1,NUMEL
127      SIGM=(TSIGEL(M,1)+TSIGEL(M,2))/2.
128      SIGD2=(TSIGEL(M,1)-TSIGEL(M,2))/2.
129      RAD=SQRT(SIGD2**2+TSIGEL(M,3)**2)
130      TSIGEP(M,1)=SIGM+RAD
131      71 TSIGEP(M,2)=SIGM-RAD
132      70 CONTINUE
133      IF(NPHASE .EQ. NPH)GO TO 87
134      WRITE(6,88)NPHASE
135      88 FORMAT(//,'PHASE',I6,/)
136      WRITE(6,72)
137      WRITE(6,76)
138      DO 89 I=1,NUMNP
139      IY=I*2
140      IX=IY-1
141      WRITE(6,77)I,TB(IX),TB(IY)
142      89 CONTINUE
143      IF(NPHASE .NE. NMATC) GO TO 87
144      DO 87 I=1,NEQ
145      TB(I)=0
146      87 CONTINUE
147      IF(NIPRIE.EQ.1)GO TO 300
148      IF(NPHASE.NE.NPH)GO TO 660
149      300 WRITE(6,60)
150      WRITE(6,61)(M,TSIGEL(M,1),TSIGEL(M,2),TSIGEL(M,3),
151      1TSIGEP(M,1),TSIGEP(M,2),M=1,NUMEL)
152      WRITE(6,72)
153      72 FORMAT(///,' TOTAL NODAL DISPLACEMENT')
154      WRITE(6,76)
155      76 FORMAT(///,' NODE NO',10X,'U',14X,'V')
156      DO 75 I=1,NUMNP
157      IY=I*2
158      IX=IY-1
159      75 WRITE(6,77)I,TB(IX),TB(IY)
160      77 FORMAT(I10,2F15.6)
161      660 K=NPHASE+1
162      DO 454 I=1,NNTP
163      J=NTP(I)
164      IY=J*2
165      IX=IY-1
166      DX=TB(IX)*50.
167      DY=TB(IY)*50.
168      XP(I,K)=X(J)+DX
169      454 YP(I,K)=Y(J)+DY
170      600 FORMAT(3I5,4F15.5)
171      C
172      C   IF ALL THE STEPS HAVE BEEN PERFORMED
173      C   PROCEED TO THE NEXT PHASE
174      C
175      GO TO 90
176      91 WRITE(6,92)
177      92 FORMAT(//////////,' UNREAL EVERYTHING WORKED !')
178      IF(1PLOT.NE.1)GO TO 455
179      CALL CGPL(XG,YG,NNG,1,1,1,4,1,-200,200,.24,
180      1-200,200,.16,TITLE,6)

```



```

181      DO 470 KL=1,NNTP
182      XPF(1)=XP(KL,1)
183      YPF(1)=YP(KL,1)
184      CALL CGPL(XPF,YPF,XPF,1,4,1,1,1,1,-200.,200.,24.,
185      1-200.,200.,16.,TITLE,6)
186 470  CONTINUE
187      NPH=NPH+1
188      DO 458 I=1,NNTP
189      DO 459 K=1,NPH
190      XPF(K)=XP(I,K)
191 459  YPF(K)=YP(I,K)
192 458  CALL CGPL(XPF,YPF,XPF,NPH,130,1,1,4,1,-200.,200.,24.,
193      1-200.,200.,16.,TITLE,6)
194      CALL CGPL(XPF,YPF,XPF,NPH,0,1,1,4,1,-200.,200.,24.,
195      1-200.,200.,16.,TITLE,6)
196 455  CONTINUE
197      STOP
198      END
199      SUBROUTINE CST
200      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
201      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
202      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
203      1 NEQ,M,LM(6)
204      COMMON KODE1(400)
205      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
206      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
207      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
208      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
209      COMMON EE(700),PRT(700),TSIGEP(700,2)
210      COMMON PHI,XK,XEP
211      COMMON DI(800)
212      REAL KO
213  C
214  C
215      CALL ASTIF
216  C
217      CALL BAND1
218  C
219      CALL STRESS
220      RETURN
221      END
222      SUBROUTINE ASTIF
223  C
224  C      THIS SUBROUTINE TAKES EACH ELEMENT IN TURN AND FORMS THE
225  C      ELEMENT STIFFNESS MATRIX (BY CALLING ELSTIF). IT ASSEMBLES
226  C      THE ELEMENT STIFFNESSES INTO KSTIF, ASSEMBLES THE APPLIED
227  C      LOAD VECTOR (AP), AND MODIFIES THE ASSEMBLAGES FOR
228  C      DISPLACEMENT BOUNDARY CONDITIONS (BY CALLING MODIFY)
229  C
230      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
231      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
232      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
233      1 NEQ,M,LM(6)
234      COMMON KODE1(400)
235      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
236      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
237      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
238      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
239      COMMON EE(700),PRT(700),TSIGEP(700,2)
240      COMMON PHI,XK,XEP

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241      COMMON DI(800)
242      REAL KO
243      C
244      C INITIALIZE APPLIED LOAD VECTOR AND MASTER STIFFNESS
245      C MATRIX AND ECM
246      C
247      DO 10 I=1,NEQ
248      AP(I)=0.0
249      DO 10 J=1,MBAND
250 10    STIF(I,J)=0.0
251      DO 20 I=1,3
252      DO 20 J=1,3
253 20    ECM(I,J)=0.0
254      C FORM ELEMENT CONSTITUTIVE MATRIX (ECM) IF NUMAT=1)
255      C
256      IF(NUMAT.NE.1) GO TO 30
257      EPR=PR(1)
258      COM=E(1)/((1.+EPR)*(1.-2.*EPR))
259      COM1=COM*(1.-EPR)
260      COM2=COM*EPR
261      ECM(1,1)=COM1
262      ECM(2,2)=COM1
263      ECM(1,2)=COM2
264      ECM(2,1)=COM2
265      ECM(3,3)=E(1)/(2.*(1.+EPR))
266      C
267 30    DO 45 M=1,NUMEL
268      C
269      CALL ELSTIF(1)
270      C
271      C ASSEMBLE ELSTIF INTO MASTER STIFFNESS MATRIX
272      C
273      DO 35 I=1,3
274      I2=2*I
275      LM(I2)=2*NP(I,M)
276 35    LM(I2-1)=LM(I2)-1
277      C
278      C
279      DO 40 I=1,6
280      II=LM(I)
281      DO 40 J=1,6
282      JJ=LM(J)-II+1
283      IF (JJ.LE.0) GO TO 40
284      STIF(II,JJ)= STIF(II,JJ)+ESTIF(I,J)
285 40    CONTINUE
286      C
287      C ADD GRAVITY LOADS INTO AP VECTOR
288      C
289      DO 45 I=2,6,2
290      II=LM(I)
291      AP(II)=AP(II)-WT
292 45    CONTINUE
293      C
294      C ADD NODAL LOADS INTO AP VECTOR
295      C
296      DO 50 N=1,NUMNP
297      N2=2*N
298      AP(N2)=AP(N2)+V(N)
299 50    AP(N2-1)=AP(N2-1)+U(N)
300      C

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301 C   MODIFY STIFFNESS AND LOAD VECTOR FOR DISPLACEMENT
302 C   BOUNDARY CONDITIONS
303 C
304     DO 100 N=1,NUMNP
305     N2=2*N
306     IF (KODE(N)-10) 80,70,60
307 60    II=N2-1
308     CALL MODIFY(II,N)
309     CALL MODIFY(N2,N)
310     GO TO 100
311 70    II=N2-1
312     CALL MODIFY(II,N)
313     GO TO 100
314 80    IF(KODE(N).EQ.0) GO TO 100
315     CALL MODIFY(N2,N)
316 100   CONTINUE
317 C
318     RETURN
319     END
320     SUBROUTINE ELSTIF(KOP)
321 C   THIS SUBROUTINE FORMS THE ELEMENT STIFFNESS MATRIX (ESTIF) OR
322 C   ELEMENT STRESS MATRIX (ESM) FOR THE CONSTANT STRAIN TRIANGLE
323 C
324     COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
325     1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
326     COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
327     1 NEQ,M,LM(6)
328     COMMON KODE1(400)
329     COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
330     COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
331     COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
332     COMMON UN(400),VN(400),NNYL(25),NNXL(25)
333     COMMON EE(700),PRT(700),TSIGEP(700,2)
334     COMMON PHI,XK,XEP
335     COMMON DI(800)
336     REAL KO
337 C
338     DIMENSION AV(3),BV(3)
339 C
340     DO 10 I=1,3
341     DO 10 J=1,6
342 10    EBM(I,J)=0.0
343     I=NP(1,M)
344     J=NP(2,M)
345     K=NP(3,M)
346 C
347 C   FORM ELEMENT DIMENSIONS
348 C
349     AV(1)=X(K)-X(J)
350     AV(2)=X(I)-X(K)
351     AV(3)=X(J)-X(I)
352     BV(1)=Y(J)-Y(K)
353     BV(2)=Y(K)-Y(I)
354     BV(3)=Y(I)-Y(J)
355     AREA2=AV(3)*BV(2)-AV(2)*BV(3)
356     IF (TH(M).EQ.0.) TH(M)=1
357     VOL=TH(M)*AREA2/2.
358     IF (VOL.LE.0.) GO TO 75
359 C   FORM CONSTITUTIVE MATRIX
360 C

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361      IF (NUMAT.EQ.1) GO TO 30
362      NM=MAT(M)
363      EPR=PRT(M)
364      COM=EE(M)/((1.+EPR)*(1.-2.*EPR))
365      COM1=COM*(1.-EPR)
366      COM2=COM*EPR
367      ECM(1,1)=COM1
368      ECM(2,2)=COM1
369      ECM(1,2)=COM2
370      ECM(2,1)=COM2
371      ECM(3,3)=EE(M)/(2.*(1.+EPR))
372  C
373  C   FORM ELEMENT B MATRIX (EBM)
374  C
375  30   DO 40 I=1,3
376       I2=2*I
377       I2M=2*I-1
378       EBM(1,I2M)=BV(I)/AREA2
379       EBM(2,I2)=AV(I)/AREA2
380       EBM(3,I2M)=EBM(2,I2)
381  40   EBM(3,I2)=EBM(1,I2M)
382  C
383  C   FORM ELEMENT STRESS MATRIX (ESM)
384  C
385       DO 50 I=1,3
386       DO 50 J=1,6
387       ESM(I,J)=0.0
388       DO 50 K=1,3
389  50   ESM(I,J)=ESM(I,J)+ECM(I,K)*EBM(K,J)
390  C
391       IF(KOP.EQ.2) GO TO 80
392       IF(NUMAT.EQ.1) NM=1
393       WT=VOL*RO(NM)/3.
394  C
395  C   FORM ELEMENT STIFFNESS MATRIX (ESTIF)
396  C
397       DO 70 I=1,6
398       DO 70 J=1,6
399       ESTIF(I,J)=0.0
400       DO 60 K=1,3
401  60   ESTIF(I,J)=ESTIF(I,J)+EBM(K,I)*ESM(K,J)
402  70   ESTIF(I,J)=ESTIF(I,J)*VOL
403       GO TO 80
404  75   WRITE(6,1000) M
405       CALL EXIT
406  80   RETURN
407  C
408  C
409  1000  FORMAT (1H1, 18H VOLUME OF ELEMENT ,I4, 18H IS LESS THAN ZERO)
410  C
411       END
412       SUBROUTINE MODIFY (I,N)
413  C
414  C   THIS SUBROUTINE MODIFIES KSTIF AND AP IF A DISPLACEMENT
415  C   BOUNDARY CONDITIONS IS IMPOSED IN EQUATION I ASSOCIATED WITH
416  C   NODAL POINT N
417  C
418       COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
419       1STIF(800,140),AP(800),ESTIF(6,6),FCM(3,3),EBM(3,6),ESM(3,6),WT
420       COMMON NUMNP,NUMEL,NUMAT,KODF(400),NP(3,700),MAT(700),MBAND,

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421      1 NEQ,M,LM(6)
422      COMMON KODE1(400)
423      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
424      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
425      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
426      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
427      COMMON EE(700),PRT(700),TSIGEP(700,2)
428      COMMON PHI,XK,XEP
429      COMMON DI(800)
430      REAL KO
431      C
432      DISP=U(N)
433      IF((I-2*N).EQ.O) DISP=V(N)
434      C
435      DO 50 J=2,MBAND
436      IL=I+J-1
437      IU=I-J+1
438      IF(IU.LE.O)GO TO 10
439      AP(IU)=AP(IU) - STIF(IU,J)*DISP
440      STIF(IU,J)=O.O
441      10 IF(IL.GT.NEQ) GO TO 50
442      AP(IL)= AP(IL)- STIF(I,J)*DISP
443      STIF(I,J)=O.O
444      50 CONTINUE
445      AP(I)=DISP
446      STIF(I,1)=1.O
447      RETURN
448      END
449      SUBROUTINE BAND1
450      C
451      C THIS SUBROUTINE SOLVES FOR DISPLACEMENTS USING A GAUSSIAN
452      C ELIMINATION TECHNIQUE FOR SYMMETRIC BANDED MATRICES STORED
453      C IN CORE
454      C
455      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
456      1 A(800,140), B(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
457      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MM,
458      1 NN,M,LM(6)
459      COMMON KODE1(400)
460      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
461      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
462      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
463      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
464      COMMON EE(700),PRT(700),TSIGEP(700,2)
465      COMMON PHI,XK,XEP
466      COMMON DI(800)
467      REAL KO
468      C
469      C TRIANGULARIZE AND REDUCE RIGHT HAND SIDE
470      NL=NN-MM+1
471      NM=NN-1
472      MR=MM
473      C
474      DO 100 N=1,NM
475      IF (A(N,1).LE.O.) GO TO 700
476      BN=B(N)
477      B(N)=BN/A(N,1)
478      IF (N.GT.NL) MR=NN-N+1
479      C
480      DO 100 L=2,MR

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481      IF (A(N,L).EQ.O.)GO TO 100
482      C= A(N,L)/A(N,1)
483      I= N+L-1
484      J= 0
485      DO 50 K=L,MR
486      J=J+1
487 50    A(I,J)= A(I,J)-C*A(N,K)
488      B(I)=B(I)-C*BN
489      A(N,L)=C
490 100   CONTINUE
491      C
492      C   BACK SUBSTITUTE
493      C
494      I=NN
495      C
496      B(NN)=B(NN)/A(NN,1)
497      DO 600 N=1,NM
498      I=I-1
499      IF (N.LT.MM)      MR= N+1
500      DO 600 J=2,MR
501      K=I+J-1
502 600    B(I) =B(I)- A(I,J)*B(K)
503      DO 30 I=1,NN
504 30    DI(I)=B(I)
505      RETURN
506 700    WRITE (6,2000) N
507      CALL EXIT
508 2000   FORMAT(1H0, 81H ZERO OR NEGATIVE ELEMENT ON  MAIN DIAGONAL OF
509      1TRIANGULARIZED MATRIX FOR EQUATION  ,I5)
510      C
511      END
512      SUBROUTINE STRESS
513      C
514      C   THIS SUBROUTINE FORMS THE ELEMENT STRESS MATRIX (ESM),MULTIPLIES
515      C   BY THE ELEMENT DISPLACEMENT VECTOR (ELDISP)AND RECORDS THE
516      C   STRESSES IN SIGEL. IT THEN COMPUTES THE PRINCIPAL STRESSES
517      C   AND DIRECTIONS (SIGP)
518      C
519      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
520      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
521      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
522      1 NEQ,M,LM(6)
523      COMMON KODE1(400)
524      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
525      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
526      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
527      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
528      COMMON EE(700),PRT(700),TSIGEP(700,2)
529      COMMON PHI,XK,XEP
530      COMMON DI(800)
531      REAL KO
532      C
533      DIMENSION SIGEL(700,3),SIGP(700,4),ELDISP(6),SIGA(400,3),
534      1 KOUNT(400)
535      EQUIVALENCE ( STIF(1,1),SIGEL(1,1)),( STIF(1,4),SIGP(1,1)),
536      1 (STIF(1,8),SIGA(1,1))
537      C
538      DO 5 N=1,NUMNP
539      KOUNT(N)=0
540      DO 5 J=1,3

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541      5      SIGA(N,J)=0.0
542      C
543      DO 100 M=1,NUMEL
544      C      COMPUTE ELEMENT DISPLACEMENTS
545      C
546          DO 10 I=1,3
547              I2=2*I
548              LMI2 =2*NP(I,M)
549              ELDISP(I2)=AP(LMI2)
550      10      ELDISP(I2-1)=AP(LMI2-1)
551      C
552      C      COMPUTE ELEMENT STRESSES
553      C
554          CALL ELSTIF(2)
555      C
556          DO 20 I=1,3
557              SIGEL(M,I)=0.0
558              DO 20 J=1,6
559      20      SIGEL(M,I)=SIGEL(M,I)+ESM(I,J)*ELDISP(J)
560      C
561      C      ACCUMULATE FOR NODAL STRESSES
562      C
563          DO 30 K=1,3
564              N=NP(K,M)
565              KOUNT(N)=KOUNT(N)+1
566              DO 30 I=1,3
567      30      SIGA(N,I)=SIGA(N,I) + SIGEL(M,I)
568      C
569      C      COMPUTE ELEMENT PRINCIPAL STRESSES AND DIRECTIONS
570      C
571          SIGM=(SIGEL(M,1)+SIGEL(M,2))/2.
572          SIGD2=(SIGEL(M,1)-SIGEL(M,2))/2.
573          RAD =SQRT(SIGD2**2 +SIGEL(M,3)**2)
574          SIGP(M,1)=SIGM+RAD
575          SIGP(M,2)= SIGM-RAD
576          SIGP(M,3)= RAD
577          IF(SIGP(M,3).LT.0.01.AND.SIGP(M,3).GT.-0.01)GO TO 700
578          SIGP(M,4)= 0.5*57.29578*ATAN2(SIGEL(M,3),SIGD2)
579          GO TO 100
580      700 SIGP(M,4)=0.
581      100 CONTINUE
582      C
583      C      FIND AVERAGE NODAL STRESSES
584      C
585          DO 110 N=1,NUMNP
586              RK=KOUNT(N)
587              DO 110 I=1,3
588      110      SIGA(N,I)= SIGA(N,I)/RK
589              DO 47 M=1,NUMEL
590                  DO 47 I=1,3
591      47      DSIGEL(M,I)=-SIGEL(M,I)
592              DO 48 N=1,NUMNP
593                  DO 48 I=1,3
594      48      DSIGA(N,I)=-SIGA(N,I)
595          END
596      C
597      C
598      C
599      C      SUBROUTINE DEFE
600      C

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601 C
602 C
603 SUBROUTINE DETE(NIPRIE,NPHASE,IJ)
604 C
605 C
606 C
607 C THIS SUBROUTINE DETERMINES E FOR EACH ELEMENT
608 C
609 C
610 C
611 COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
612 1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
613 COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
614 1 NEQ,M,LM(6)
615 COMMON KODE1(400)
616 COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
617 COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
618 COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
619 COMMON UN(400),VN(400),NNYL(25),NNXL(25)
620 COMMON EE(700),PRT(700),TSIGEP(700,2)
621 COMMON PHI,XK,XEP
622 COMMON DI(800)
623 REAL KO
624 EP1(SIG3,SIG1,AA,BB)=AA*(SIG1-SIG3)/(1.-(SIG1-SIG3)*BB)
625 E1(AA,BB,FEP1,PR)=2.*PR*AA*100./(AA+BB*FEP1)**2
626 EP2(SIG3,SIG1,SIG3I,SIG1I,AA,BB)=AA*((SIG3-SIG1)-
627 1(SIG3I-SIG1I))/(1.-((SIG3-SIG1)-(SIG3I-SIG1I))*BB)
628 E2(AA,BB,FEP2)=AA*100./(AA+BB*FEP2)**2
629 EP5(SIG3,SIG1,SIG3I,SIG1I,AA,BB)=AA*((SIG1-SIG3)-
630 1(SIG1I-SIG3I))/(1.-((SIG1-SIG3)-(SIG1I-SIG3I))*BB)
631 E5(AA,BB,FEP5,P)=(AA*100./(AA+BB*FEP5)**2)*P*(1.+P)
632 EP7(SIG3,SIG1,AA,BB)=AA*(SIG1-SIG3)/(1.-(SIG1-SIG3)*BB)
633 E7(AA,BB,FEP7)=AA*100./(AA+BB*FEP7)**2
634 IF(NIPRIE.EQ.1)GO TO 450
635 IF(NPHASE.NE.7)GO TO 451
636 IF(IJ.NE.4)GO TO 451
637 450 WRITE(6,105)
638 105 FORMAT(1H1,5X,'NEW STRESS STRAIN PARAMETERS',//,1X,
639 1' EL MAT SIGX IN SIGY IN SIG1 SIG3',
640 26X,'SIGX SIGY ETANGENT')
641 451 CONTINUE
642 DO 999 M=1,NUMEL
643 C
644 C PRINCIPAL STRESSFS
645 C
646 S2=TSIGEP(M,2)
647 S1=TSIGEP(M,1)
648 S3=S1-S2
649 T1=SIGIEL(M,1)
650 T2=SIGIEL(M,2)
651 C
652 C MATERIAL NUMBER 1
653 C
654 IF(MAT(M).NE.1)GO TO 1
655 PRT(M)=PR(1)
656 V1=SIGC(1,1)/KO(1)
657 V2=SIGC(1,2)/KO(1)
658 V3=SIGC(1,3)/KO(1)
659 C
660 C CHECKING WHERE SIGMA3 INITIAL STANDS

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661      C
662      IF(SIGIEL(M,1).LT.SIGC(1,1))GO TO 83
663      IF(SIGIEL(M,1).GT.SIGC(1,3)) GO TO 81
664      IF(SIGIEL(M,1).GT.SIGC(1,2)) GO TO 82
665      C
666      C SU ALWAYS IS A VALUE OF DEVIATOR STRESS ABOVE
667      C WHICH IT IS TOO CLOSE TO FAILURE.
668      C IF THAT HAPPENS E VALUE IS ASSUMED A VERY SMALL
669      C VALUE TO AVOID PROBLEMS WHEN CALCULATING EPSILON1
670      C
671      SU=0.95/BB(1,1)
672      AS=SIGC(1,1)
673      AB=SIGC(1,2)
674      IF(S3.GT.SU) GO TO 41
675      XS=EP1(S2,S1,AA(1,1),BB(1,1))
676      ES=E1(AA(1,1),BB(1,1),XS,PR(1))
677      100 FORMAT(5F15.5)
678      GO TO 42
679      41 ES=20.
680      42 SU=0.95/BB(1,2)
681      IF(S3.GT.SU) GO TO 43
682      XB=EP1(S2,S1,AA(1,2),BB(1,2))
683      EB=E1(AA(1,2),BB(1,2),XB,PR(1))
684      GO TO 99
685      43 EB=20.
686      C
687      C
688      C INTERPOLATING E VALUE
689      C IF SIGMA3 INITIAL SMALLER THAN THE ANY OF THE TESTS
690      C USE STRESS STRAIN CURVE WHERE A=A(1,1)
691      C B FUNCTION ON PHI AND SIGMA VERTICAL
692      C FOR THIS PARTICULAR CASE SIN(PHI)=.642
693      C IF SIGMA3 INITIAL GREATER THAN ANY OF THE TESTS
694      C USE STRESS STRAIN CURVE FOR THE GREATEST SIGMA3
695      C
696      C
697      99 EE(M)=(EB*(SIGIEL(M,1)-AS)+ES*(AB-SIGIEL(M,1)))/(AB-AS)
698      GO TO 10
699      82 SU=0.95/BB(1,2)
700      AS=SIGC(1,2)
701      AB=SIGC(1,3)
702      IF(S3.GT.SU) GO TO 44
703      XS=EP1(S2,S1,AA(1,2),BB(1,2))
704      ES=E1(AA(1,2),BB(1,2),XS,PR(1))
705      GO TO 45
706      44 ES=20.
707      45 SU=0.95/BB(1,3)
708      IF(S3.GT.SU)GO TO 46
709      XB=EP1(S2,S1,AA(1,3),BB(1,3))
710      EB=E1(AA(1,3),BB(1,3),XB,PR(1))
711      GO TO 99
712      46 EB=20.
713      GO TO 99
714      81 SU=0.95/BB(1,3)
715      IF(S3.GT.SU)GO TO 40
716      XX=EP1(S2,S1,AA(1,3),BB(1,3))
717      EE(M)=E1(AA(1,3),BB(1,3),XX,PR(1))
718      GO TO 10
719      83 BE=(1+.642)/(2*.642*T2)
720      SU=.95/BE

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721         IF(S3.GT.SU) GO TO 40
722         XX=EP1(S2,S1,AA(1,1),BE)
723         EE(M)=E1(AA(1,1),BE,XX,PR(1))
724         GO TO 10
725     40 EE(M)=20.
726         GO TO 10
727     C
728     C   MATERIAL NUMBER 2
729     C
730     1  IF(MAT(M).NE.2)GO TO 2
731         PRT(M)=PR(2)
732         S3=-S3
733         IF(TSIGEL(M,2).GT.TSIGEL(M,1)) GO TO 101
734         TEMP=S2
735         S2=S1
736         S1=TEMP
737         S3=-S3
738     101 V1=SIGC(2,1)/KO(2)
739         V2=SIGC(2,2)/KO(2)
740         V3=SIGC(2,3)/KO(2)
741     C
742     C   CHECKING WHERE SIGMA3 INITIAL STANDS
743     C
744         IF(SIGIEL(M,1).LT.SIGC(2,1))GO TO 13
745         IF(SIGIEL(M,1).GT.SIGC(2,3))GO TO 11
746         IF(SIGIEL(M,1).GT.SIGC(2,2)) GO TO 12
747     C
748     C   SIGMA3 IN BETWEEN TESTS 1 AND 2
749     C
750         SU=0.95*((T1-T2)+1./BB(2,1))
751         AS=SIGC(2,1)
752         AB=SIGC(2,2)
753         IF(S3.GT.SU) GO TO 48
754         XS=EP2(S2,S1,T1,T2,AA(2,1),BB(2,1))
755         IF(XS)321,321,322
756     321 XS=0.
757     322 ES=E2(AA(2,1),BB(2,1),XS)
758         GO TO 47
759     48 ES=20.
760     47 SU=0.95*((T1-T2)+1./BB(2,2))
761         IF(S3.GT.SU) GO TO 58
762         XB=EP2(S2,S1,T1,T2,AA(2,2),BB(2,2))
763         IF(XB)323,323,324
764     323 XB=0.
765     324 EB=E2(AA(2,2),BB(2,2),XB)
766         GO TO 99
767     58 EB=20.
768         GO TO 99
769     C
770     C   SIGMA3 IN BETWEEN TESTS 2 AND 3
771     C
772     12 SU=0.95*((T1-T2)+1./BB(2,2))
773         AS=SIGC(2,2)
774         AB=SIGC(2,3)
775         IF(S3.GT.SU) GO TO 55
776         XS=EP2(S2,S1,T1,T2,AA(2,2),BB(2,2))
777         IF(XS)325,325,326
778     325 XS=0.
779     326 ES=E2(AA(2,2),BB(2,2),XS)
780         GO TO 56

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781      55 ES=20.
782      56 SU=0.95*((T1-T2)+1./BB(2,3))
783          IF(S3.GT.SU) GO TO 57
784          XB=EP2(S2,S1,T1,T2,AA(2,3),BB(2,3))
785          IF(XB)327,327,328
786      327 XB=0.
787      328 EB=E2(AA(2,3),BB(2,3),XB)
788          GO TO 99
789      57 EB=20.
790          GO TO 99
791  C
792  C   SIGMA3 GREATER THAN TEST 3
793  C
794      11 SU=0.95*((T1-T2)+1./BB(2,3))
795          IF(S3.GT.SU) GO TO 40
796          XX=EP2(S2,S1,T1,T2,AA(2,3),BB(2,3))
797          IF(XX)329,329,330
798      329 XX=0.
799      330 EE(M)=E2(AA(2,3),BB(2,3),XX)
800          GO TO 10
801  C
802  C   SIGMA3 SMALLER THAN TEST 1
803  C
804      13 SU=0.95*((T1-T2)+1./BB(2,1))
805          IF(S3.GT.SU) GO TO 40
806          XX=EP2(S2,S1,T1,T2,AA(2,1),BB(2,1))
807          IF(XX)331,331,332
808      331 XX=0.
809      332 EE(M)=E2(AA(2,1),BB(2,1),XX)
810          GO TO 10
811  C
812  C   MATERIAL NUMBER 3
813  C
814      2  IF(MAT(M).NE.3) GO TO 3
815          V1=SIGIEL(M,2)
816          PRT(M)=PR(3)
817          T3=(T2-T1)*0.95
818          IF(S3.LT.T3) GO TO 31
819  C
820  C   IF DEVIATOR STRESS GREATER THAN INITIAL DEVIATOR STRESS
821  C   USE STRESS STRAIN CURVE DISPLACED BY INITIAL DEVIATOR STRESS
822  C
823          SU=1.00*((T2-T1)+1./BB(3,1))
824          IF(S3.GT.SU) GO TO 61
825          XX=EP2(S1,S2,T2,T1,AA(3,1),BB(3,1))
826          EE(M)=E1(AA(3,1),BB(3,1),XX,PR(3))
827          GO TO 10
828      61 EE(M)=20.
829          GO TO 10
830  C
831  C   IF DEVIATOR STRESS LESS THAN INITIAL DEVIATOR STRESS
832  C   USE E FOR LOADING UNLOADING ACCORDING TO THIS STRESS PATH
833  C
834      31 EE(M)=E(3)
835          GO TO 10
836  C
837  C   MATERIAL NUMBER 4
838  C
839      3  IF(MAT(M).NE.4) GO TO 4
840          EE(M)=E(4)

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841      PRT(M)=PR(4)
842      GO TO 10
843  C
844  C   MATERIAL NUMBER 5
845  C
846      4  IF(MAT(M).NE.5) GO TO 5
847      PRT(M)=PR(5)
848      V1=SIGC(5,1)/KO(5)
849      V2=SIGC(5,2)/KO(5)
850      IF(SIGIEL(M,1).LT.SIGC(5,1)) GO TO 51
851      IF(SIGIEL(M,1).GT.SIGC(5,2)) GO TO 51
852  C
853  C   INTERPOLATION OF E
854  C
855      SU=0.95*((T2-T1)+1./BB(5,1))
856      AS=SIGC(5,1)
857      AB=SIGC(5,2)
858      IF(S3.GT.2.) GO TO 63
859      IF(S2.LT.0.)GO TO 63
860      XS=EP5(S2,S1,T1,T2,AA(5,1),BB(5,1))
861      IF(XS)400,400,401
862      400 XS=0.
863      401 ES=E5(AA(5,1),BB(5,1),XS,PR(5))
864      GO TO 64
865      63 ES=20.
866      64 SU=0.95*((T2-T1)+1./BB(5,2))
867      IF(S3.GT.2.5)GO TO 65
868      IF(S2.LT.0.)GO TO 65
869      XB=EP5(S2,S1,T1,T2,AA(5,2),BB(5,2))
870      IF(XB)402,402,403
871      402 XB=0.
872      403 EB=E5(AA(5,2),BB(5,2),XB,PR(5))
873      GO T099
874      65 EB=20.
875      GO TO 99
876  C
877  C
878  C   IF SIGMA3 GREATER THAN THE GREATEST OF SIGMA3 IN THE
879  C   TESTS ASSUME THE SAME STRESS STRAIN CURVE SHIFTED
880  C   BY INITIAL DEVIATOR STRESS
881  C
882  C
883      51  IF(S2.LT.0.)GO TO 62
884      SU=T2
885      IF(S3.GT.SU)GO TO 62
886      EY01=7.696*(T1/1.033)*0.9436
887      AE=1./EY01
888      BE=(0.25+AE*(T2-T1-T2))/(.25*(T2-(T2-T1)))
889      XX=EP5(S2,S1,T1,T2,AE,BE)
890      IF(XX)404,404,405
891      404 XX=0.
892      405 EE(M)=E5(AE,BE,XX,PR(5))
893      GO TO 10
894      62 EE(M)=20.
895      GO TO 10
896  C
897  C   MATERIAL NUMBER 6
898  C
899      5  IF(MAT(M).NE.6)GO TO 6
900      EE(M)=E(6)

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901      PRT(M)=PR(6)
902      GOTO 10
903      C
904      C MATERIAL NUMBER 7
905      C
906      6 IF(MAT(M).NE.7) GO TO 7
907      PRT(M)=PR(7)
908      C
909      C CHECKING WHERE SIGMA3 STANDS
910      C
911      IF(S2.LT.SIGC(7,1))GO TO 73
912      IF(S2.GT.SIGC(7,3)) GO TO 71
913      IF(S2.GT.SIGC(7,2))GO TO 72
914      C
915      C IF SIGMA3 IS IN BETWEEN TETS 1 AND 2
916      C
917      SU=0.95/BB(7,1)
918      AS=SIGC(7,1)
919      AB=SIGC(7,2)
920      IF(S3.GT.SU) GO TO 66
921      XS=EP7(S2,S1,AA(7,1),BB(7,1))
922      IF(XS)301,301,302
923      301 XS=0.
924      302 ES=E7(AA(7,1),BB(7,1),XS)
925      GO TO 67
926      66 ES=20.
927      67 SU=0.95/BB(7,2)
928      IF(S3.GT.SU) GO TO 68
929      XB=EP7(S2,S1,AA(7,2),BB(7,2))
930      IF(XB)303,303,304
931      303 XB=0.
932      304 EB=E7(AA(7,2),BB(7,2),XB)
933      GO TO 89
934      68 EB=20.
935      GO TO 89
936      C
937      C IF SIGMA3 STANDS IN BETWEEN TESTS 2 AND 3
938      C
939      72 SU=0.95/BB(7,2)
940      AS=SIGC(7,2)
941      AB=SIGC(7,3)
942      IF(S3.GT.SU) GO TO 84
943      XS=EP7(S2,S1,AA(7,2),BB(7,2))
944      IF(XS)305,305,306
945      305 XS=0
946      306 ES=E7(AA(7,2),BB(7,2),XS)
947      GO TO 85
948      84 ES=20.
949      85 SU=0.95/BB(7,3)
950      IF(S3.GT.SU) GO TO 86
951      XB=EP7(S2,S1,AA(7,3),BB(7,3))
952      IF(XB)307,307,308
953      307 XB=0.
954      308 EB=E7(AA(7,3),BB(7,3),XB)
955      GO TO 89
956      86 EB=20.
957      GO TO 89
958      C
959      C IF SIGMA3 GREATER THAN IN TEST 3
960      C

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961      71 SU=0.95/BB(7,3)
962      IF(S3.GT.SU) GO TO 95
963      XX=EP7(S2,S1,AA(7,3),BB(7,3))
964      IF(XX)309,309,310
965      309 XX=0.
966      310 EE(M)=E7(AA(7,3),BB(7,3),XX)
967      GO TO 10
968      C
969      C IF SIGMA3 SMALLER THAN IN TEST 1
970      C USE STRESS STRAIN CURVE WHERE
971      C B FUNCTION OF PHI AND SIGMA HORIZONTAL
972      C A FUNCTION OF SIGMA3 , XK AND XEP
973      C
974      73 IF(S2.LT.0.5)GO TO 95
975      AE=1./((XK*1.033*(S2/1.033)**XEP)
976      BE=(1.-SIN(PHI))/(2.*S2*SIN(PHI))
977      SU=0.95/BE
978      IF(S3.GT.SU) GO TO 95
979      XX=EP7(S2,S1,AE,BE)
980      IF(XX)311,311,312
981      311 XX=0.
982      312 EE(M)=(E7(AE,BE,XX))/100.
983      GO TO 10
984      95 EE(M)=20.
985      GO TO 10
986      89 EE(M)=(EB*(TSIGEP(M,2)-AS)+ES*(AB-TSIGEP(M,2)))
987      1/(AB-AS)
988      GO TO 10
989      C
990      C MATERIAL NUMBER 8
991      C
992      7 IF(MAT(M).NE.8) GO TO 8
993      PRT(M)=PR(8)
994      EE(M)=E(8)
995      GO TO 10
996      C
997      C MATERIAL NUMBER 9
998      C
999      8 IF(MAT(M).NE.9) GO TO 200
1000      IF(S2.LT.0.2)GO TO 103
1001      PRT(M)=PR(9)
1002      BE=(1.-1.143*SIN(PHI))/(2.*S2*1.143*SIN(PHI))
1003      SU=0.75/BE
1004      IF(S3.GT.SU)GO TO 103
1005      XX=EP7(S2,S1,AA(9,1),BE)
1006      EE(M)=E7(AA(9,1),BE,XX)
1007      GO TO 10
1008      103 EE(M)=20.
1009      GO TO 10
1010      200 PRT(M)=PR(10)
1011      EE(M)=E(10)
1012      IF(TSIGEL(M,1).LT.0.)EE(M)=10.
1013      10 IF(NIPRIE.EQ.1)GO TO 998
1014      IF(NPHASE.NE.7)GO TO 999
1015      998 IF(IJ.NE.4)GO TO 999
1016      WRITE(6,93) M,MAT(M),SIGIEL(M,1),SIGIEL(M,2),TSIGEP(M,1),
1017      1TSIGEP(M,2),TSIGEL(M,1),TSIGEL(M,2),EF(M)
1018      93 FORMAT(1X,2I5,6F10.3,F14.3)
1019      999 CONTINUE
1020      RETURN

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1021      END
1022      C
1023      C
1024      C
1025      C   SUBROUTINE INIT
1026      C
1027      C
1028      C
1029      C   SUBROUTINE INIT
1030      C
1031      C
1032      C   THIS SUBROUTINE CALCULATES THE INITIAL STRESSES
1033      C   AT THE NODES AND ELEMENTS
1034      C
1035      C
1036      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
1037      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
1038      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
1039      1 NEQ,M,LM(6)
1040      COMMON KODE1(400)
1041      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
1042      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
1043      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
1044      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
1045      COMMON EE(700),PRT(700),TSIGEP(700,2)
1046      COMMON PHI,XK,XEP
1047      COMMON DI(800)
1048      REAL KO
1049      C
1050      C   READ SPECIFIC WEIGHT AND ELEVATION OF THE GROUND SURFACE
1051      C
1052      READ(5,13)GAMA,ZSFC
1053      WRITE(6,10)GAMA,ZSFC
1054      10 FORMAT(//,5X,'SPECIFIC WEIGHT ',F10.5,/,5X,'HEIGHT',F10.3)
1055      13 FORMAT(2F10.3)
1056      C
1057      C   IF THE MATERIAL ELEMENT IS CONCRETE
1058      C   SET INITIAL STRESSES = 0
1059      C
1060      DO 1 M=1,NUMEL
1061      IF(MAT(M).NE.4) GO TO 2
1062      SIGIEL(M,1)=0.
1063      SIGIEL(M,2)=0.
1064      SIGIEL(M,3)=0.
1065      GO TO 1
1066      C
1067      C   INITIAL ELEMENT STRESSES
1068      C
1069      2   I=NP(1,M)
1070      J=NP(2,M)
1071      K=NP(3,M)
1072      YEL=(Y(I)+Y(J)+Y(K))/3.
1073      DEPTH=ZSFC-YEL
1074      SIGIEL(M,2)=GAMA*DEPTH
1075      MATN=MAT(M)
1076      SIGIEL(M,1)=SIGIEL(M,2)*KO(MATN)
1077      SIGIEL(M,3)=0.
1078      1   CONTINUE
1079      C
1080      C   INITIAL NODE STRESSES

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1081      C
1082      DO 3 M=1,NUMNP
1083      IF(KODE1(M).NE.4) GO TO 4
1084      SIGIA(M,1)=0.
1085      SIGIA(M,2)=0.
1086      SIGIA(M,3)=0.
1087      GO TO 3
1088      4 DEPTH=ZSFC-Y(M)
1089      SIGIA(M,2)=GAMA*DEPTH
1090      MATN=KODE1(M)
1091      SIGIA(M,1)=SIGIA(M,2)*KO(MATN)
1092      SIGIA(M,3)=SIGIA(M,2)-SIGIA(M,1)
1093      3 CONTINUE
1094      RETURN
1095      END
1096      C
1097      C
1098      C
1099      C SUBROUTINE NLOAD
1100      C
1101      C
1102      C
1103      SUBROUTINE NLOAD(NPHASE,NIPRIN,NPH)
1104      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
1105      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
1106      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
1107      1 NEQ,M,LM(6)
1108      COMMON KODE1(400)
1109      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
1110      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
1111      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
1112      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
1113      COMMON EE(700),PRT(700),TSIGEP(700,2)
1114      COMMON PHI,XK,XEP
1115      COMMON DI(800)
1116      DIMENSION SIGNR(400,2),IEL(20),UA(400),VA(400)
1117      C
1118      C
1119      C
1120      C THIS SUBROUTINE DETERMINES THE NEW LOADS DUE TO ANOTHER
1121      C CONSTRUCTION PHASE. IT CHANGES THE MATERIAL PROPERTIES FOR
1122      C THE MATERIAL WHICH HAS BEEN REMOVED OR CONCRETE WHICH
1123      C HAS BEEN POURED
1124      C
1125      C
1126      C
1127      DO 35 I=1,NUMNP
1128      SIGNR(I,1)=0
1129      SIGNR(I,2)=0
1130      UN(I)=0.
1131      35 VN(I)=0.
1132      READ(5,1)NPHASE,NYL,NXL
1133      IF(NPHASE.GT.NPH) RETURN
1134      WRITE(6,2)NPHASE
1135      2 FORMAT(1H1,/,5(1X,60(' '),/),2(1X,20(' '),20X,20(' '),/),
1136      11X,20(' '), ' PHASE NUMBER',13,2X,20(' '),/,2(1X,
1137      220(' '),20X,20(' '),/),5(1X,60(' '),/),////////)
1138      C
1139      C NYL NUMBER OF NODES LOADED VERTICALLY
1140      C NNYL NODE NUMBER WHICH WILL BE LOADED

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1141 C   NELI NUMBER OF ELEMENTS INVOLVED IN THIS NODE
1142 C
1143     DO 40 I=1,NYL
1144         READ(5,1)NELI,NNYL(I)
1145         READ(5,1)(IEL(J),J=1,NELI)
1146         NN=NNYL(I)
1147         DO 41 K=1,NELI
1148             41  SIGNR(NN,2)=SIGNR(NN,2)+TSIGEL(IEL(K),2)
1149             XN=NELI
1150             SIGNR(NN,2)=SIGNR(NN,2)/XN
1151             WRITE(6,42)NN,SIGNR(NN,2)
1152             42  FORMAT(//,'  NODE NO ',I5,'  SIGY REMOVED',F10.5)
1153             WRITE(6,60)
1154             60  FORMAT(/,'  ELEMENTS INVOLVED')
1155             WRITE(6,3)(IEL(J),J=1,NELI)
1156             40  CONTINUE
1157 C
1158 C   NXL NUMBER OF NODES LOADED HORIZONTALLY
1159 C   NNXL NODE NUMBER WHICH WILL BE LOADED
1160 C   NELI NUMBER OF ELEMENTS INVOLVED IN THIS NODE
1161 C
1162     DO 50 I=1,NXL
1163         READ(5,1)NELI,NNXL(I)
1164         READ(5,1)(IEL(J),J=1,NELI)
1165         NN=NNXL(I)
1166         DO 51 K=1,NELI
1167             51  SIGNR(NN,1)=SIGNR(NN,1)+TSIGEL(IEL(K),1)
1168             XN=NELI
1169             SIGNR(NN,1)=SIGNR(NN,1)/XN
1170             WRITE(6,52)NN,SIGNR(NN,1)
1171             52  FORMAT(//,'  NODE NO ',I5,'  SIGX REMOVED',F10.5)
1172             WRITE(6,60)
1173             WRITE(6,3)(IEL(J),J=1,NELI)
1174             50  CONTINUE
1175             1  FORMAT(10I5)
1176             WRITE(6,80)
1177             80  FORMAT(/,5X,'NODES LOADED HORIZ')
1178             WRITE(6,3)(NNXL(I),I=1,NXL)
1179             3  FORMAT(/,5X,5I7)
1180             WRITE(6,4)
1181             4  FORMAT(/,5X,'NODES LOADED VERT')
1182             WRITE(6,3)(NNYL(I),I=1,NYL)
1183             DO 5 I=1,NUMNP
1184 C
1185 C   LOADS IN THE X DIRECTION
1186 C
1187     DO 6 J=1,NXL
1188         XLOAD=0.
1189         IF(I.EQ.NNXL(J))GO TO 7
1190         GO TO 6
1191 C
1192 C   CHECK IF IT IS THE LAST NODE AFFECTED
1193 C
1194     7  IF(J.EQ.NXL) GO TO 9
1195 C
1196         K=NNXL(J+1)
1197         DL=Y(K)-Y(I)
1198 C
1199 C   CHECK IF THE ADJACENT NODE HAS HIGHER STRESSES
1200 C

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1201      IF(SIGNR(K,1).GT.SIGNR(I,1)) GO TO 10
1202      XB=SIGNR(I,1)
1203      XS=SIGNR(K,1)
1204      XLOAD=XS*DL/2.+2./3.*0.5*(XB-XS)*DL
1205      XLOAD=XLOAD*TH(1)
1206      GO TO 9
1207 10    XB=SIGNR(K,1)
1208      XS=SIGNR(I,1)
1209      XLOAD=XS*DL/2.+1./3.*0.5*(XB-XS)*DL
1210      XLOAD=XLOAD*TH(1)
1211  C
1212  C    CHECK IF IT IS THE FIRST NODE AFFECTED
1213  C
1214 9     IF(J.EQ.1) GO TO 12
1215      K=NNXL(J-1)
1216      DL=Y(I)-Y(K)
1217  C
1218  C    CHECK IF THE ADJACENT NODE HAS HIGHER STRESSES
1219  C
1220      IF(SIGNR(K,1).GT.SIGNR(I,1)) GO TO 11
1221      XB=SIGNR(I,1)
1222      XS=SIGNR(K,1)
1223      XLOAD=XLOAD+(XS*DL/2.+2./3.*0.5*(XB-XS)*DL)*TH(1)
1224      GO TO 12
1225 11    XB=SIGNR(K,1)
1226      XS=SIGNR(I,1)
1227      XLOAD=XLOAD+(XS*DL/2.+1./3.*.5*(XB-XS)*DL)*TH(1)
1228 12    UN(I)=-XLOAD*1.1
1229 6     CONTINUE
1230  C
1231  C    LOADS IN THE Y DIRECTION
1232  C
1233      DO 13 J=1,NYL
1234      YLOAD=0.
1235      IF(I.EQ.NNYL(J)) GO TO 14
1236  C
1237  C    CHECK IF IT IS THE LAST NODE AFFECTED
1238  C
1239      GO TO 13
1240 14    IF(J.EQ.NYL) GO TO 17
1241      K=NNYL(J+1)
1242      DL=X(K)-X(I)
1243  C
1244  C    CHECK IF THE ADJACENT NODE HAS HIGHER STRESSES
1245  C
1246      IF(SIGNR(K,2).GT.SIGNR(I,2)) GO TO 16
1247      YB=SIGNR(I,2)
1248      YS=SIGNR(K,2)
1249      YLOAD=YS*DL/2.+2./3.*0.5*(YB-YS)*DL
1250      YLOAD=YLOAD*TH(1)
1251      GO TO 17
1252 16    YB=SIGNR(K,2)
1253      YS=SIGNR(I,2)
1254      YLOAD=YS*DL/2.+1./3.*0.5*(YB-YS)*DL
1255      YLOAD=YLOAD*TH(1)
1256 17    IF(J.EQ.1) GO TO 18
1257      K=NNYL(J-1)
1258      DL=X(I)-X(K)
1259  C
1260  C    CHECK IF THE ADJACENT NODE HAS HIGHER STRESSES

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```

1261      C
1262      IF(SIGNR(K,2).GT.SIGNR(I,2)) GO TO 19
1263      YB=SIGNR(I,2)
1264      YS=SIGNR(K,2)
1265      YLOAD=YLOAD+(YS*DL/2.+2./3.*0.5*(YB-YS)*DL)*TH(1)
1266      GO TO 18
1267      19 YB=SIGNR(K,2)
1268      YS=SIGNR(I,2)
1269      YLOAD=YLOAD+(YS*DL/2.+1./3.*0.5*(YB-YS)*DL)*TH(1)
1270      18 VN(I)=YLOAD
1271      13 CONTINUE
1272      5 CONTINUE
1273      C
1274      C IF THERE IS ANY ANCHOR FORCE
1275      C INPUT NUMBER OF NODES LOADED
1276      C NAF=NUMBER OF NODES THAT ANCHOR FORCE APPLIED
1277      C NNAF=NODE NUMBER
1278      C UA=X FORCE
1279      C VA=Y FORCE
1280      C
1281      READ(5,1)NAF
1282      IF(NAF.EQ.0) GO TO 110
1283      WRITE(6,103)NAF
1284      WRITE(6,104)
1285      102 FORMAT(I10,4F15.2)
1286      103 FORMAT(//.5X,'NUMBER OF TIE BACK FORCES=',I6)
1287      104 FORMAT(//.6X,'NODE',8X,'X FORCE',8X,'Y FORCE',8X,
1288      1'X FORCE',8X,'Y FORCE')
1289      DO 110 I=1,NAF
1290      READ(5,102)NNAF,UA(NNAF),VA(NNAF)
1291      UN(NNAF)=UN(NNAF)+UA(NNAF)
1292      VN(NNAF)=VN(NNAF)+VA(NNAF)
1293      WRITE(6,102)NNAF,UA(NNAF),VA(NNAF),UN(NNAF),VN(NNAF)
1294      110 CONTINUE
1295      C
1296      C
1297      C IF THERE IS ANY TRANSFER OF NODAL LOADS
1298      C DUE TO GEOMETRICAL CONFIGURATION ENTER NLD
1299      C NUMBER OF NODES WHICH WILL HAVE THE LOAD
1300      C TRANSFERRED OTHERWISE
1301      C
1302      C
1303      READ(5,1)NLD
1304      WRITE(6,501)NLD
1305      501 FORMAT(//.5X,'NUMBER OF NODES WHICH WILL HAVE ',
1306      1'THE LOAD TRANSFERRED',I5)
1307      IF(NLD.EQ.0)GO TO 502
1308      READ(5,1)NA,NB
1309      WRITE(6,503)NA,NB
1310      503 FORMAT(//.5X,'NODES CARRYING THE LOAD',2I10)
1311      DO 504 I=1,NLD
1312      READ(5,1)NU
1313      WRITE(6,505)NU
1314      505 FORMAT(//.5X,'NODE UNLOADED',I5)
1315      C
1316      C
1317      C TRANSFER THE LOAD OF NODE NU TO NODES NA NB
1318      C Y(NA) HAS TO BE GREATER THAN Y(NB)
1319      C
1320      C

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```

1321      XXA=Y(NA)-Y(NU)
1322      XXL=Y(NA)-Y(NB)
1323      RB=UN(NU)*XXA/XXL
1324      RA=UN(NU)-RB
1325      UN(NU)=0.
1326      UN(NA)=UN(NA)+RA
1327      UN(NB)=UN(NB)+RB
1328      504  FORMAT(4F10.5)
1329      506  WRITE(6,21)(I,UN(I),VN(I),I=1,NUMNP)
1330      502  IF(NIPRIN.NE.1)GO TO 666
1331      WRITE(6,20)NPHASE
1332      20  FORMAT(/,5X,'NODAL LOAD FOR PHASE NUMBER',I5,
1333      1//,5X,'NODE HORIZONTAL  VERTICAL ')
1334      WRITE(6,21)(I,UN(I),VN(I),I=1,NUMNP)
1335      21  FORMAT(5X,I5,2F10.5)
1336      C
1337      C
1338      C  ELEMENTS TO BE CHANGED
1339      C
1340      C
1341      666  READ(5,22)NELCA
1342      WRITE(6,26)
1343      26  FORMAT(/,5X,'ELEMENTS WHICH WILL BE REMOVED')
1344      22  FORMAT(I5)
1345      IF(NELCA.EQ.0) GO TO 27
1346      DO 23 I=1,NELCA
1347      READ(5,22)INEL
1348      WRITE(6,22)INEL
1349      TSIGEL(INEL,1)=0.
1350      TSIGEL(INEL,2)=0.
1351      TSIGEL(INEL,3)=0.
1352      23  MAT(INEL)=8
1353      27  READ(5,22)NELCC
1354      WRITE(6,28)
1355      28  FORMAT(/,5X,'ELEMENTS WHICH WILL BE CHANGED TO CONCRETE ')
1356      IF(NELCC.EQ.0)GO TO 31
1357      DO 24 I=1,NELCC
1358      READ(5,33)INEL,TH(INEL)
1359      WRITE(6,33)INEL,TH(INEL)
1360      33  FORMAT(I6,F15.4)
1361      MAT(INEL)=4
1362      TSIGEL(INEL,1)=0.
1363      TSIGEL(INEL,2)=0.
1364      24  TSIGEL(INEL,3)=0.
1365      C
1366      C  NODES THAT WILL CHANGE THE BOUNDARY CONDITION
1367      C
1368      READ(5,22)NNBC
1369      IF(NNBC.EQ.0)GOTO 65
1370      DO 65 I=1,NNBC
1371      READ(5,64)NNCC,KODE(NNCC)
1372      64  FORMAT(2I5)
1373      65  CONTINUE
1374      READ(5,22)NELCG
1375      WRITE(6,70)
1376      70  FORMAT(/,5X,'ELEMENTS WHICH WILL BE GROUTED')
1377      IF(NELCG.EQ.0) GO TO 31
1378      DO 71 I=1,NELCG
1379      READ(5,22)INEL
1380      TSIGEL(INEL,1)=0

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```

1381      TSIGEL(INEL,2)=0.
1382      TSIGEL(INEL,3)=0.
1383      WRITE(6,22)INEL
1384 71    MAT(INEL)=10
1385 C
1386 C
1387 C   NODES WHICH WILL HAVE THE STRESSES ZEROED
1388 C   CONCRETE JUST POURED OR CONCRETE SOIL REMOVED INTERFACE NODES
1389 C
1390 C
1391      31 WRITE(6,29)
1392      29 FORMAT(/,5X,'NODES WHICH WILL HAVE THE STRESSES ZEROED')
1393      READ(5,22)NNC
1394      IF(NNC.EQ.0)GO TO 32
1395      DO 25 I=1,NNC
1396      READ(5,22)J
1397      WRITE(6,22)J
1398      TSIGA(J,1)=0.
1399      TSIGA(J,2)=0.
1400      25 TSIGA(J,3)=0.
1401      32 CONTINUE
1402 C
1403 C   ELEMENTS THAT CHANGE MATERIAL PROPERTIES (NELCM)
1404 C
1405 C   ELEMENT MATERIAL NUMBER (NMATN)
1406 C
1407      READ (5,22) NELCM
1408      WRITE (6,100)
1409      100 FORMAT (/,5X,' ELEMENTS WHICH CHANGE PROPERTIES ')
1410      IF (NELCM.EQ.0) GO TO 101
1411      READ (5,22) NMATN
1412      DO 101 I=1,NELCM
1413      READ (5,22)INEL
1414      WRITE (6,22) INEL
1415      MAT(INEL)=NMATN
1416      101 CONTINUE
1417      RETURN
1418      END
1419 C
1420      SUBROUTINE SST
1421 C
1422 C   THIS SUBROUTINE READS STRESS STRAIN PARAMETERS
1423 C
1424      COMMON E(10),PR(10),RO(10),X(400),Y(400),U(400),V(400),TH(700),
1425      1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
1426      COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MEAND,
1427      1 NEQ,M,LM(6)
1428      COMMON KODE1(400)
1429      COMMON SIGC(10,4),AA(10,4),BB(10,4),KO(10),ZSFC
1430      COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
1431      COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
1432      COMMON UN(400),VN(400),NNYL(25),NNXL(25)
1433      COMMON EE(700),PRT(700),TSIGEP(700,2)
1434      COMMON PHI,XK,XEP
1435      COMMON DI(800)
1436      REAL KO
1437 C
1438 C   READ AND WRITE STRESS STRAIN PARAMETERS
1439 C
1440      WRITE(6,48)

```



```

1441      48  FORMAT(1H1)
1442      10  READ(5,1)MM
1443      1   FORMAT(I5)
1444
1445  C      POISSONS RATIO AND KO
1446  C
1447      READ(5,4)PR(MM),KO(MM)
1448      4   FORMAT(2F10.5)
1449      IF(MM-1)13,13,3
1450
1451  C      MATERIAL NUMBER 1
1452  C      3 TESTS TRIAXIAL STRESS PATH COMP ACTIVE
1453  C
1454      13  DO 5 J=1,3
1455      5   READ(5,6)SIGC(MM,J),AA(MM,J),BB(MM,J)
1456          WRITE(6,7)MM,PR(MM),KO(MM)
1457      7   FORMAT(///,5X,'MATERIAL NUMBER',I5,///,5X,'POISSONS RATIO ',
1458          1F10.2,/,5X,'KO= ',F10.2)
1459          WRITE(6,9)
1460      9   FORMAT(5X,'CONF STR          A          B')
1461          WRITE(6,8)(SIGC(MM,J),AA(MM,J),BB(MM,J),J=1,3)
1462      8   FORMAT(5X,3F10.6)
1463          GO TO 10
1464      3   IF(MM-2)11,11,12
1465
1466  C      MATERIAL NUMBER 2
1467  C      3 TESTS TRIAXIAL STRESS PATH EXTENSION ACTIVE
1468  C
1469      11  READ(5,6)(SIGC(MM,J),AA(MM,J),BB(MM,J),J=1,3)
1470          WRITE(6,7)(MM,PR(MM),KO(MM))
1471          WRITE(6,9)
1472          WRITE(6,8)(SIGC(MM,J),AA(MM,J),BB(MM,J),J=1,3)
1473          GO TO 10
1474      12  IF(MM-3)16,16,17
1475
1476  C      MATERIAL NUMBER 3
1477  C      1 TEST TRIAXIAL STRESS PATH KO-COMP ACTIVE
1478  C
1479      16  READ(5,6)E(MM),AA(MM,1),BB(MM,1)
1480          WRITE(6,15)MM,PR(MM),E(MM),AA(MM,1),BB(MM,1),KO(MM)
1481      15  FORMAT(///,5X,'MATERIAL NUMBER',I5,///,5X,'POISSONS RATIO ',
1482          1F10.2,/,5X,'E UNLOAD-RELOADING ',F10.3,///,5X,'A ',F10.6,
1483          2/,5X,'B ',F10.6,/,5X,'KO' ,F10.2)
1484      6   FORMAT(3F10.6)
1485          GO TO 10
1486      17  IF(MM-4)18,18,19
1487
1488  C      MATERIAL NUMBER 4
1489  C      1 TEST CONCRETE
1490  C
1491      18  READ(5,6)E(MM)
1492          WRITE(6,21)MM,E(MM),PR(MM),KO(MM)
1493      21  FORMAT(///,5X,'MATERIAL NUMBER ',I5,/,5X,'E ',F15.5,/,
1494          15X,'POISSONS RATIO ',F10.5,/,5X,'KO', F10.2)
1495          GO TO 10
1496      19  IF(MM-5)22,22,23
1497
1498  C      MATERIAL NUMBER 5
1499  C      2 TESTS PLANE STRAIN COMP. ACTIVE
1500  C

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1501      22 DO 24 J=1,2
1502      24 READ(5,6)SIGC(MM,J),AA(MM,J),BB(MM,J)
1503          WRITE(6,7)MM,PR(MM),KO(MM)
1504          WRITE(6,9)
1505          WRITE(6,8)(SIGC(MM,J),AA(MM,J),BB(MM,J),J=1,2)
1506          GO TO 10
1507      23 IF(MM-6)25,25,26
1508      C
1509      C MATERIAL NUMBER 6
1510      C 1 TEST CONVENTIONAL TRIAXIAL FOR FILL
1511      C
1512          25 READ(5,4)E(MM)
1513              WRITE(6,7)MM,PR(MM),KO(MM)
1514              WRITE(6,28)E(MM)
1515          28 FORMAT(5X,'E',F15.5)
1516          GO TO 10
1517      26 IF(MM-7)36,36,27
1518      C
1519      C MATERIAL NUMBER 7
1520      C 3 TESTS TRIAXIAL CONVENTIONAL TRIAXIAL
1521      C
1522          36 DO 30 J=1,3
1523      30 READ(5,6)SIGC(MM,J),AA(MM,J),BB(MM,J)
1524          WRITE(6,7)MM,PR(MM),KO(MM)
1525          WRITE(6,9)
1526          WRITE(6,8)(SIGC(MM,J),AA(MM,J),BB(MM,J),J=1,3)
1527          READ(5,6)PHI,XK,XEP
1528          WRITE(6,49)PHI,XK,XEP
1529          49 FORMAT(5X,'PHI=',F10.3,/,5X,'K=',F10.2,/,5X,'N=',F10.2)
1530          GO TO 10
1531      27 IF(MM-8)37,37,38
1532      C
1533      C MATERIAL NUMBER 8
1534      C AIR
1535      C
1536          37 READ(5,6)E(MM)
1537              WRITE(6,34)MM,E(MM),PR(MM),KO(MM)
1538          34 FORMAT(///,5X,'MATERIAL NUMBER',I5,/,5X,'E',F10.2,
1539              1/,5X,'PR',F10.3,/,5X,'KO',F10.2)
1540          GO TO 10
1541          38 IF(MM-9)52,52,53
1542      C
1543      C MATERIAL NUMBER 9
1544      C 1 TEST CONVENTIONAL TRIAXIAL
1545      C
1546          52 READ(5,6)SIGC(MM,1),AA(MM,1),BB(MM,1)
1547              WRITE(6,7)MM,PR(MM),KO(MM)
1548              WRITE(6,9)
1549              WRITE(6,8)SIGC(MM,1),AA(MM,1),BB(MM,1)
1550              GO TO 10
1551          53 IF(MM-10)250,250,251
1552      250 READ(5,6)E(MM)
1553          WRITE(6,21)MM,E(MM),PR(MM),KO(MM)
1554          RETURN
1555          251 WRITE(6,31)
1556          31 FORMAT(///,5X,'ERROR TOO MANY MATERIALS')
1557          RETURN
1558          END
1559      C
1560      C

```



```

1561 C
1562 C SUBROUTINE READGE
1563 C
1564 C
1565 C
1566 SUBROUTINE READGE
1567 C
1568 C THIS SUBROUTINE READS AND PRINTS MATERIAL DATA, NODAL DATA,
1569 C ELEMENT DATA. IT GENERATES COORDINATES OF INTERMEDIATE NODAL
1570 C POINTS AND CALCULATES THE BAND WIDTH AND NUMBER OF EQUATIONS
1571 C
1572 C
1573 C
1574 COMMON E(10),PR(10),RD(10),X(400),Y(400),U(400),V(400),TH(700),
1575 1STIF(800,140),AP(800),ESTIF(6,6),ECM(3,3),EBM(3,6),ESM(3,6),WT
1576 COMMON NUMNP,NUMEL,NUMAT,KODE(400),NP(3,700),MAT(700),MBAND,
1577 1 NEQ,M,LM(6)
1578 COMMON KODE1(400)
1579 COMMON SIGC(10,4),AA(10,4),BB(10,4),KD(10),ZSFC
1580 COMMON DSIGEL(700,3),SIGIEL(700,3),DSIGA(400,3),SIGIA(400,3)
1581 COMMON TSIGEL(700,3),TSIGA(400,3),TB(800)
1582 COMMON UN(400),VN(400),NNYL(25),NNXL(25)
1583 COMMON EE(700),PRT(700),TSIGEP(700,2)
1584 COMMON PHI,XK,XEP
1585 COMMON DI(800)
1586 REAL KO
1587 C
1588 DIMENSION HED(20)
1589 C READ PRELIMINARY INFORMATION
1590 READ(5,1000) HED,NUMNP,NUMEL,NUMAT
1591 WRITE(6,2000) HED,NUMNP,NUMEL,NUMAT
1592 C
1593 C READ AND WRITE NODAL DATA AND GENERATE INTERMEDIATE NODAL DATA
1594 C
1595 L=1
1596 READ(5,1020) N,KODE(N),X(N),Y(N),U(N),V(N),KODE1(N)
1597 WRITE(6,2020)N,KODE(N),X(N),Y(N),U(N),V(N),KODE1(N)
1598 GO TO 40
1599 C
1600 20 READ(5,1020) N,KODE(N),X(N),Y(N),U(N),V(N),KODE1(N)
1601 WRITE(6,2020)N,KODE(N),X(N),Y(N),U(N),V(N),KODE1(N)
1602 DN = N-L
1603 DX =(X(N)-X(L))/DN
1604 DY =(Y(N)-Y(L))/DN
1605 25 L=L+1
1606 C
1607 IF(N-L) 50,40,30
1608 30 X(L) = X(L-1)+DX
1609 Y(L) = Y(L-1)+DY
1610 KODE1(L)=KODE1(L-1)
1611 KODE(L)= 0
1612 U(L) = 0
1613 V(L)= 0
1614 GO TO 25
1615 C
1616 40 CONTINUE
1617 IF(NUMNP-N) 50,60,20
1618 C
1619 50 WRITE (6,2025) N
1620 CALL EXIT

```



```

1621      C
1622      60 WRITE(6,2016)
1623          WRITE(6,2015)
1624          WRITE (6,2020) (N,KODE(N),X(N),Y(N),U(N),V(N),KODE1(N),N=1,NUMNP)
1625      C
1626      C READ AND WRITE ELEMENT DATA
1627      C
1628          ML=0
1629      51  IF (ML.GE.NUMEL) GO TO 70
1630          READ(5,1035) M,NP(1,M),NP(2,M),NP(3,M),MAT(M),TH(M)
1631          WRITE(6,2035) M,NP(1,M),NP(2,M),NP(3,M),MAT(M),TH(M)
1632          MM=ML+1
1633          IF (MM.EQ.M) GO TO 65
1634      C
1635      55  ML1=ML+1
1636          IF(ML1.EQ.M) GO TO 65
1637          ML2=ML+2
1638          MLM1=ML-1
1639          IF(MLM1.LE.0) GO TO 85
1640          DO 62 I=1,3
1641              NP(I,ML2)=NP(I,ML)+1
1642      62  NP(I,ML1)=NP(I,MLM1)+1
1643          MAT(ML2)=MAT(ML)
1644          MAT(ML1)=MAT(MLM1)
1645          TH(ML2)=TH(ML)
1646          TH(ML1)=TH(MLM1)
1647          ML=ML2
1648          GO TO 55
1649      C
1650      65  ML=M
1651          GO TO 51
1652      70  CONTINUE
1653          WRITE(6,2032)
1654          WRITE(6,2030)
1655          WRITE (6,2035) (M,(NP(J,M),J=1,3),MAT(M),TH(M),M=1,NUMEL)
1656      C
1657      C DETERMINE BAND WIDTH AND NUMBER OF EQUATIONS
1658      C
1659          L=0
1660          DO 80 M=1,NUMEL
1661              DO 80 I=1,2
1662                  II=I+1
1663                  DO 80 J=II,3
1664                      K= IABS(NP(I,M)-NP(J,M))
1665                      IF (K.GT.L) L=K
1666      80  CONTINUE
1667      C
1668          MBAND = 2*(L+1)
1669          NEQ= 2*NUMNP
1670      C
1671          WRITE(6,2040)MBAND,NEQ
1672          IF(MBAND.LE. 140.AND.NEQ.LE.800) GO TO 90
1673          WRITE(6,2050)
1674          CALL EXIT
1675      85  WRITE(6,2060)
1676          CALL EXIT
1677      C
1678      90  WRITE(6,3000)
1679      3000 FORMAT( ' READIN COMPLETED ' ///)
1680      RETURN

```



```

1681 C
1682 C  FORMAT STATEMENTS
1683 1000  FORMAT(20A4/ 3I6)
1684 2000  FORMAT(/,10X,20A4,////
1685      1 1H ,      26H NUMBER OF NODAL POINTS = .I6/
1686      2 1H ,      26H NUMBER OF ELEMENTS    = .I6/
1687      3 1H ,      26H NUMBER OF MATERIALS    = .I6)
1688 1010  FORMAT(6X,F12.0,2F6.0)
1689 2010  FORMAT(1H ,I5,F17.0,F15.3,F17.1)
1690 2014  FORMAT( /, 5X, 'OUTPUT OF INPUT NODAL DATA ')
1691 2015  FORMAT(///,10X,19H NODAL POINT OUTPUT,///
1692      11H ,59H NODE  KODE      X COORD      Y COORD      X FORCE      Y FORCE
1693      2//)
1694 2016  FORMAT('1', 5X, 'OUTPUT OF COMPLETE NODAL DATA ')
1695 1020  FORMAT(2I6,4F12.0,I5)
1696 2020  FORMAT(I4,I6,F13.3,3F12.3,I5)
1697 2025  FORMAT(1H0,28H ERROR IN NODAL DATA,NODE = ,I4)
1698 2030  FORMAT(///,10X, 13H ELEMENT DATA ///,
1699      1 40H ELEM  I      J      K      MAT THICKNESS //)
1700 2031  FORMAT('1',5X, 'OUTPUT OF INPUT ELEMENT DATA' )
1701 2032  FORMAT( '1', 5X, 'OUTPUT OF COMPLETE ELEMENT DATA ' )
1702 1035  FORMAT (5I6,F6.0)
1703 2035  FORMAT ( I4, 4I6,F11.4)
1704 2040  FORMAT (///10X, 22H BAND WIDTH          = .I6/
1705      1      10X, 22H NUMBER OF EQUATIONS = .I6)
1706 2050  FORMAT(///10X, 33H PROBLEM EXCEEDS SPECIFIED LIMITS )
1707 2060  FORMAT( 'MLM1 IS LESS THAN OR EQUAL TO ZERO ')
1708 C
1709      END

```


input data

0	0	0	0			
107	ST	STATION				
372	672	10				
1	11	0.0	0 0	0.0	0.0	9
2	10	0.0	200.000	0.0	0.0	9
3	10	0.0	700.000	0.0	0.0	7
4	10	0.0	970.000	0.0	0.0	7
5	10	0.0	1240.000	0.0	0.0	7
6	10	0.0	1357.500	0.0	0.0	9
7	10	0.0	1475.000	0.0	0.0	7
8	10	0.0	1530.000	0.0	0.0	7
9	10	0.0	1600.000	0.0	0.0	7
10	10	0.0	1750.000	0.0	0.0	7
11	10	0.0	1900.000	0.0	0.0	7
12	10	0.0	2030.000	0.0	0.0	7
13	10	0.0	2080.000	0.0	0.0	7
14	10	0.0	2130.000	0.0	0.0	7
15	10	0.0	2150.000	0.0	0.0	7
16	10	0.0	2250.000	0.0	0.0	7
17	10	0.0	2400.000	0.0	0.0	7
18	10	0.0	2500.000	0.0	0.0	7
19	10	0.0	2590.000	0.0	0.0	7
20	10	0.0	2650.000	0.0	0.0	7
21	10	0.0	2720.000	0.0	0.0	7
22	10	0.0	2785.000	0.0	0.0	7
23	10	0.0	2800.000	0.0	0.0	7
24	11	150.000	0.0	0.0	0.0	9
25	0	150.000	200.000	0.0	0.0	9
26	0	150.000	700.000	0.0	0.0	7
27	0	150.000	970.000	0.0	0.0	7
28	0	150.000	1240.000	0.0	0.0	9
30	0	150.000	1475.000	0.0	0.0	9
31	0	150.000	1530.000	0.0	0.0	7
32	0	150.000	1600.000	0.0	0.0	7
33	0	150.000	1750.000	0.0	0.0	7
34	0	150.000	1900.000	0.0	0.0	7
35	0	150.000	2030.000	0.0	0.0	7
37	0	150.000	2130.000	0.0	0.0	7
38	0	150.000	2150.000	0.0	0.0	7
39	0	150.000	2250.000	0.0	0.0	7
40	0	150.000	2400.000	0.0	0.0	7
41	0	150.000	2500.000	0.0	0.0	7
42	0	150.000	2590.000	0.0	0.0	7
43	0	150.000	2650.000	0.0	0.0	7
44	0	150.000	2720.000	0.0	0.0	7
45	0	150.000	2785.000	0.0	0.0	7
46	0	150.000	2800.000	0.0	0.0	7
47	11	300.000	0.0	0.0	0.0	9
48	0	300.000	200.000	0.0	0.0	9
49	0	300.000	700.000	0.0	0.0	7
50	0	300.000	970.000	0.0	0.0	7
51	0	300.000	1240.000	0.0	0.0	9
53	0	300.000	1475.000	0.0	0.0	9
54	0	300.000	1530.000	0.0	0.0	7
55	0	300.000	1600.000	0.0	0.0	7
56	0	300.000	1750.000	0.0	0.0	7
57	0	300.000	1900.000	0.0	0.0	7
58	0	300.000	2030.000	0.0	0.0	7
60	0	300.000	2130.000	0.0	0.0	7
61	0	300.000	2150.000	0.0	0.0	7

62	0	300.000	2250.000	0.0	0.0	7
63	0	300.000	2400.000	0.0	0.0	7
64	0	300.000	2500.000	0.0	0.0	7
65	0	300.000	2590.000	0.0	0.0	7
66	0	300.000	2650.000	0.0	0.0	7
67	0	300.000	2720.000	0.0	0.0	7
68	0	300.000	2785.000	0.0	0.0	7
69	0	300.000	2800.000	0.0	0.0	7
70	11	430.000	0.0	0.0	0.0	9
71	0	430.000	200.000	0.0	0.0	9
72	0	430.000	700.000	0.0	0.0	7
73	0	430.000	970.000	0.0	0.0	7
74	0	430.000	1240.000	0.0	0.0	9
76	0	430.000	1475.000	0.0	0.0	9
77	0	430.000	1530.000	0.0	0.0	7
78	0	430.000	1600.000	0.0	0.0	7
79	0	430.000	1750.000	0.0	0.0	7
80	0	430.000	1900.000	0.0	0.0	7
81	0	430.000	2030.000	0.0	0.0	7
83	0	430.000	2130.000	0.0	0.0	7
84	0	430.000	2150.000	0.0	0.0	7
85	0	430.000	2250.000	0.0	0.0	7
86	0	430.000	2400.000	0.0	0.0	7
87	0	430.000	2500.000	0.0	0.0	7
88	0	430.000	2590.000	0.0	0.0	7
89	0	430.000	2650.000	0.0	0.0	7
90	0	430.000	2720.000	0.0	0.0	7
91	0	430.000	2785.000	0.0	0.0	7
92	0	430.000	2800.000	0.0	0.0	7
93	11	550.000	0.0	0.0	0.0	9
94	0	550.000	200.000	0.0	0.0	9
95	0	550.000	700.000	0.0	0.0	7
96	0	550.000	970.000	0.0	0.0	7
97	0	550.000	1240.000	0.0	0.0	9
99	0	550.000	1475.000	0.0	0.0	9
100	0	550.000	1530.000	0.0	0.0	7
101	0	550.000	1600.000	0.0	0.0	7
102	0	550.000	1750.000	0.0	0.0	7
103	0	550.000	1900.000	0.0	0.0	7
104	0	550.000	2030.000	0.0	0.0	7
106	0	550.000	2130.000	0.0	0.0	7
107	0	550.000	2150.000	0.0	0.0	7
108	0	550.000	2250.000	0.0	0.0	7
109	0	550.000	2400.000	0.0	0.0	7
110	0	550.000	2500.000	0.0	0.0	7
111	0	550.000	2590.000	0.0	0.0	7
112	0	550.000	2650.000	0.0	0.0	7
113	0	550.000	2720.000	0.0	0.0	7
114	0	550.000	2785.000	0.0	0.0	7
115	0	550.000	2800.000	0.0	0.0	7
116	11	670.000	0.0	0.0	0.0	9
117	0	670.000	200.000	0.0	0.0	9
118	0	670.000	700.000	0.0	0.0	7
119	0	670.000	970.000	0.0	0.0	7
120	0	670.000	1240.000	0.0	0.0	9
122	0	670.000	1475.000	0.0	0.0	9
123	0	670.000	1530.000	0.0	0.0	7
124	0	670.000	1600.000	0.0	0.0	7
125	0	670.000	1750.000	0.0	0.0	7
126	0	670.000	1900.000	0.0	0.0	7

127	0	670.000	2030.000	0.0	0.0	7
129	0	670.000	2130.000	0.0	0.0	7
130	0	670.000	2150.000	0.0	0.0	7
131	0	670.000	2250.000	0.0	0.0	7
132	0	670.000	2400.000	0.0	0.0	7
133	0	670.000	2500.000	0.0	0.0	7
134	0	670.000	2590.000	0.0	0.0	7
135	0	670.000	2650.000	0.0	0.0	7
136	0	670.000	2720.000	0.0	0.0	7
137	0	670.000	2785.000	0.0	0.0	7
138	0	670.000	2800.000	0.0	0.0	7
139	11	740.000	0.0	0.0	0.0	9
140	0	740.000	200.000	0.0	0.0	9
141	0	740.000	700.000	0.0	0.0	7
142	0	740.000	970.000	0.0	0.0	7
143	0	740.000	1240.000	0.0	0.0	9
145	0	740.000	1475.000	0.0	0.0	9
146	0	740.000	1530.000	0.0	0.0	7
147	0	740.000	1600.000	0.0	0.0	7
149	0	740.000	1900.000	0.0	0.0	7
150	0	740.000	2030.000	0.0	0.0	7
152	0	740.000	2130.000	0.0	0.0	7
153	0	740.000	2150.000	0.0	0.0	7
154	0	740.000	2250.000	0.0	0.0	7
155	0	740.000	2400.000	0.0	0.0	7
156	0	740.000	2500.000	0.0	0.0	7
157	0	740.000	2590.000	0.0	0.0	7
158	0	740.000	2650.000	0.0	0.0	7
159	0	740.000	2720.000	0.0	0.0	7
160	0	740.000	2785.000	0.0	0.0	7
161	0	740.000	2800.000	0.0	0.0	7
162	11	810.000	0.0	0.0	0.0	9
163	0	810.000	200.000	0.0	0.0	9
164	0	810.000	700.000	0.0	0.0	7
165	0	810.000	970.000	0.0	0.0	7
166	0	810.000	1240.000	0.0	0.0	9
168	0	810.000	1475.000	0.0	0.0	9
169	0	810.000	1530.000	0.0	0.0	7
170	0	810.000	1600.000	0.0	0.0	7
172	0	810.000	1900.000	0.0	0.0	7
173	0	810.000	2030.000	0.0	0.0	7
175	0	810.000	2130.000	0.0	0.0	7
176	0	810.000	2150.000	0.0	0.0	7
177	0	810.000	2250.000	0.0	0.0	7
178	0	810.000	2400.000	0.0	0.0	7
179	0	810.000	2500.000	0.0	0.0	7
180	0	810.000	2590.000	0.0	0.0	7
181	0	810.000	2650.000	0.0	0.0	7
182	0	810.000	2720.000	0.0	0.0	7
183	0	810.000	2785.000	0.0	0.0	7
184	0	810.000	2800.000	0.0	0.0	7
185	11	900.000	0.0	0.0	0.0	9
186	0	900.000	200.000	0.0	0.0	9
187	0	900.000	700.000	0.0	0.0	7
189	0	900.000	970.000	0.0	0.0	7
191	0	900.000	1240.000	0.0	0.0	9
193	0	900.000	1475.000	0.0	0.0	9
194	0	900.000	1530.000	0.0	0.0	7
195	0	900.000	1600.000	0.0	0.0	7
197	0	900.000	1900.000	0.0	0.0	7

198	0	900.000	2030.000	0.0	0.0	7
200	0	900.000	2130.000	0.0	0.0	7
201	0	900.000	2150.000	0.0	0.0	7
202	0	900.000	2250.000	0.0	0.0	7
203	0	900.000	2400.000	0.0	0.0	7
204	0	900.000	2500.000	0.0	0.0	7
205	0	900.000	2590.000	0.0	0.0	7
206	0	900.000	2650.000	0.0	0.0	7
207	0	900.000	2720.000	0.0	0.0	7
208	0	900.000	2785.000	0.0	0.0	7
209	0	900.000	2800.000	0.0	0.0	7
210	11	950.000	0.0	0.0	0.0	9
211	0	950.000	200.000	0.0	0.0	9
212	11	950.000	700.000	0.0	0.0	4
214	1	950.000	970.000	0.0	0.0	4
215	1	950.000	1105.000	0.0	0.0	4
216	1	950.000	1240.000	0.0	0.0	4
218	1	950.000	1475.000	0.0	0.0	4
219	1	950.000	1530.000	0.0	0.0	4
220	0	950.000	1600.000	0.0	0.0	4
222	0	950.000	1900.000	0.0	0.0	4
223	0	950.000	2030.000	0.0	0.0	4
225	0	950.000	2130.000	0.0	0.0	4
226	0	950.000	2150.000	0.0	0.0	4
227	0	950.000	2250.000	0.0	0.0	4
228	0	950.000	2400.000	0.0	0.0	4
229	0	950.000	2500.000	0.0	0.0	4
230	0	950.000	2590.000	0.0	0.0	4
231	0	1000.000	2650.000	0.0	0.0	7
232	0	1000.000	2720.000	0.0	0.0	7
233	0	1000.000	2785.000	0.0	0.0	7
234	0	1000.000	2800.000	0.0	0.0	7
235	11	1000.000	0.0	0.0	0.0	9
236	0	1000.000	200.000	0.0	0.0	9
237	0	1000.000	700.000	0.0	0.0	7
239	0	1000.000	970.000	0.0	0.0	7
241	0	1000.000	1240.000	0.0	0.0	9
243	0	1000.000	1475.000	0.0	0.0	9
244	0	1000.000	1530.000	0.0	0.0	7
245	0	1000.000	1600.000	0.0	0.0	7
247	0	1000.000	1900.000	0.0	0.0	7
248	0	1000.000	2030.000	0.0	0.0	7
250	0	1000.000	2130.000	0.0	0.0	7
251	0	1000.000	2150.000	0.0	0.0	7
252	0	1000.000	2250.000	0.0	0.0	7
253	0	1000.000	2400.000	0.0	0.0	7
254	0	1000.000	2500.000	0.0	0.0	7
255	0	1000.000	2590.000	0.0	0.0	7
256	11	1050.000	0.0	0.0	0.0	9
257	0	1050.000	200.000	0.0	0.0	9
258	0	1050.000	700.000	0.0	0.0	7
260	0	1050.000	970.000	0.0	0.0	7
262	0	1050.000	1240.000	0.0	0.0	9
264	0	1050.000	1475.000	0.0	0.0	9
265	0	1050.000	1530.000	0.0	0.0	7
266	0	1050.000	1600.000	0.0	0.0	7
268	0	1050.000	1900.000	0.0	0.0	7
269	0	1050.000	2030.000	0.0	0.0	7
271	0	1050.000	2130.000	0.0	0.0	7
272	0	1050.000	2150.000	0.0	0.0	7

273	0	1050.000	2250.000	0.0	0.0	7
274	0	1050.000	2400.000	0.0	0.0	7
275	0	1050.000	2500.000	0.0	0.0	7
276	0	1050.000	2590.000	0.0	0.0	7
277	0	1050.000	2650.000	0.0	0.0	7
278	0	1050.000	2720.000	0.0	0.0	7
279	0	1050.000	2800.000	0.0	0.0	7
280	11	1150.000	0.0	0.0	0.0	9
281	0	1150.000	200.000	0.0	0.0	9
282	0	1150.000	700.000	0.0	0.0	7
283	0	1150.000	970.000	0.0	0.0	7
284	0	1150.000	1240.000	0.0	0.0	9
285	0	1150.000	1475.000	0.0	0.0	9
286	0	1150.000	1600.000	0.0	0.0	7
287	0	1150.000	1900.000	0.0	0.0	7
288	0	1150.000	2080.000	0.0	0.0	7
289	0	1150.000	2250.000	0.0	0.0	7
290	0	1150.000	2400.000	0.0	0.0	7
291	0	1150.000	2590.000	0.0	0.0	7
292	0	1150.000	2800.000	0.0	0.0	7
293	11	1350.000	0.0	0.0	0.0	9
294	0	1350.000	200.000	0.0	0.0	9
295	0	1350.000	700.000	0.0	0.0	7
297	0	1350.000	1240.000	0.0	0.0	9
298	0	1350.000	1475.000	0.0	0.0	9
299	0	1350.000	1900.000	0.0	0.0	7
300	0	1350.000	2250.000	0.0	0.0	7
301	0	1350.000	2590.000	0.0	0.0	7
302	0	1350.000	2800.000	0.0	0.0	7
303	11	1650.000	0.0	0.0	0.0	9
304	0	1650.000	200.000	0.0	0.0	9
305	0	1650.000	700.000	0.0	0.0	7
307	0	1650.000	1240.000	0.0	0.0	9
308	0	1650.000	1475.000	0.0	0.0	9
309	0	1650.000	1900.000	0.0	0.0	7
310	0	1650.000	2250.000	0.0	0.0	7
311	0	1650.000	2590.000	0.0	0.0	7
312	0	1650.000	2800.000	0.0	0.0	7
313	11	2000.000	0.0	0.0	0.0	9
314	0	2000.000	200.000	0.0	0.0	9
315	0	2000.000	700.000	0.0	0.0	7
317	0	2000.000	1240.000	0.0	0.0	9
318	0	2000.000	1475.000	0.0	0.0	9
319	0	2000.000	1900.000	0.0	0.0	7
320	0	2000.000	2250.000	0.0	0.0	7
321	0	2000.000	2590.000	0.0	0.0	7
322	0	2000.000	2800.000	0.0	0.0	7
323	11	2500.000	0.0	0.0	0.0	9
324	0	2500.000	200.000	0.0	0.0	9
325	0	2500.000	700.000	0.0	0.0	7
327	0	2500.000	1240.000	0.0	0.0	9
328	0	2500.000	1475.000	0.0	0.0	9
329	0	2500.000	1900.000	0.0	0.0	7
330	0	2500.000	2250.000	0.0	0.0	7
331	0	2500.000	2590.000	0.0	0.0	7
332	0	2500.000	2800.000	0.0	0.0	7
333	11	3000.000	0.0	0.0	0.0	9
334	0	3000.000	200.000	0.0	0.0	9
335	0	3000.000	700.000	0.0	0.0	7
337	0	3000.000	1240.000	0.0	0.0	9

338	0	3000.000	1475.000	0.0	0.0	9
339	0	3000.000	1900.000	0.0	0.0	7
340	0	3000.000	2250.000	0.0	0.0	7
341	0	3000.000	2590.000	0.0	0.0	7
342	0	3000.000	2800.000	0.0	0.0	7
343	11	3500.000	0.0	0.0	0.0	9
344	0	3500.000	200.000	0.0	0.0	9
345	0	3500.000	700.000	0.0	0.0	7
347	0	3500.000	1240.000	0.0	0.0	9
348	0	3500.000	1475.000	0.0	0.0	9
349	0	3500.000	1900.000	0.0	0.0	7
350	0	3500.000	2250.000	0.0	0.0	7
351	0	3500.000	2590.000	0.0	0.0	7
352	0	3500.000	2800.000	0.0	0.0	7
353	11	4000.000	0.0	0.0	0.0	9
354	0	4000.000	200.000	0.0	0.0	9
355	0	4000.000	700.000	0.0	0.0	7
357	0	4000.000	1240.000	0.0	0.0	9
358	0	4000.000	1475.000	0.0	0.0	9
359	0	4000.000	1900.000	0.0	0.0	7
360	0	4000.000	2250.000	0.0	0.0	7
361	0	4000.000	2590.000	0.0	0.0	7
362	0	4000.000	2800.000	0.0	0.0	7
363	11	4600.000	0.0	0.0	0.0	9
364	10	4600.000	200.000	0.0	0.0	9
365	10	4600.000	700.000	0.0	0.0	7
366	10	4600.000	970.000	0.0	0.0	7
367	10	4600.000	1240.000	0.0	0.0	9
368	10	4600.000	1475.000	0.0	0.0	9
369	10	4600.000	1900.000	0.0	0.0	7
370	10	4600.000	2250.000	0.0	0.0	7
371	10	4600.000	2590.000	0.0	0.0	7
372	10	4600.000	2800.000	0.0	0.0	7
1	1	24	25	2	1.0000	
2	1	25	2	2	1.0000	
3	2	25	26	2	1.0000	
4	2	26	3	2	1.0000	
9	5	28	29	2	1.0000	
10	5	29	6	2	1.0000	
13	7	30	31	1	1.0000	
14	7	31	8	1	1.0000	
37	19	42	43	1	1.0000	
38	19	43	20	1	1.0000	
43	22	45	46	1	1.0000	
44	22	46	23	1	1.0000	
45	24	47	48	2	1.0000	
46	24	48	25	2	1.0000	
47	25	48	49	2	1.0000	
48	25	49	26	2	1.0000	
53	28	51	52	2	1.0000	
54	28	52	29	2	1.0000	
57	30	53	54	1	1.0000	
58	30	54	31	1	1.0000	
87	45	68	69	1	1.0000	
88	45	69	46	1	1.0000	
89	47	70	71	2	1.0000	
90	47	71	48	2	1.0000	
91	48	71	72	2	1.0000	
92	48	72	49	2	1.0000	
97	51	74	75	2	1.0000	

98	51	75	52	2	1.0000
101	53	76	77	1	1.0000
102	53	77	54	1	1.0000
131	68	91	92	1	1.0000
132	68	92	69	1	1.0000
133	70	93	94	2	1.0000
134	70	94	71	2	1.0000
135	71	94	95	2	1.0000
136	71	95	72	2	1.0000
141	74	97	98	2	1.0000
142	74	98	75	2	1.0000
145	76	99	100	1	1.0000
146	76	100	77	1	1.0000
175	91	114	115	1	1.0000
176	91	115	92	1	1.0000
177	93	116	117	2	1.0000
178	93	117	94	2	1.0000
179	94	117	118	2	1.0000
180	94	118	95	2	1.0000
185	97	120	121	2	1.0000
186	97	121	98	2	1.0000
189	99	122	123	1	1.0000
190	99	123	100	1	1.0000
219	114	137	138	1	1.0000
220	114	138	115	1	1.0000
221	116	139	140	2	1.0000
222	116	140	117	2	1.0000
223	117	140	141	2	1.0000
224	117	141	118	2	1.0000
229	120	143	144	2	1.0000
230	120	144	121	2	1.0000
233	122	145	146	1	1.0000
234	122	146	123	1	1.0000
263	137	160	161	1	1.0000
264	137	161	138	1	1.0000
265	139	162	163	2	1.0000
266	139	163	140	2	1.0000
267	140	163	164	2	1.0000
268	140	164	141	2	1.0000
273	143	166	167	2	1.0000
274	143	167	144	2	1.0000
275	145	168	169	1	1.0000
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395	205	230	231	4	0.7156
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671	361	371	372	1	1.0000
672	361	372	362	1	1.0000

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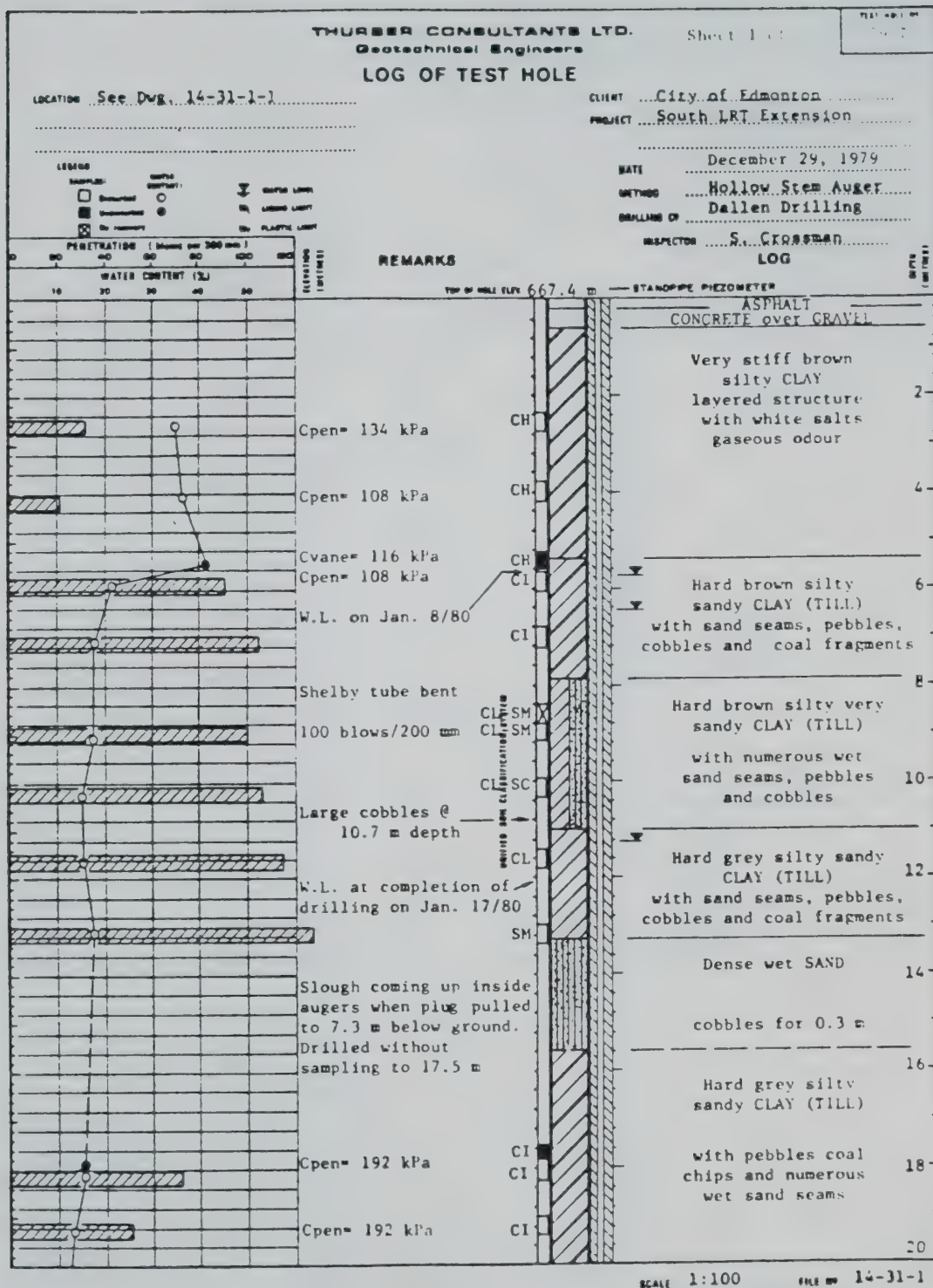
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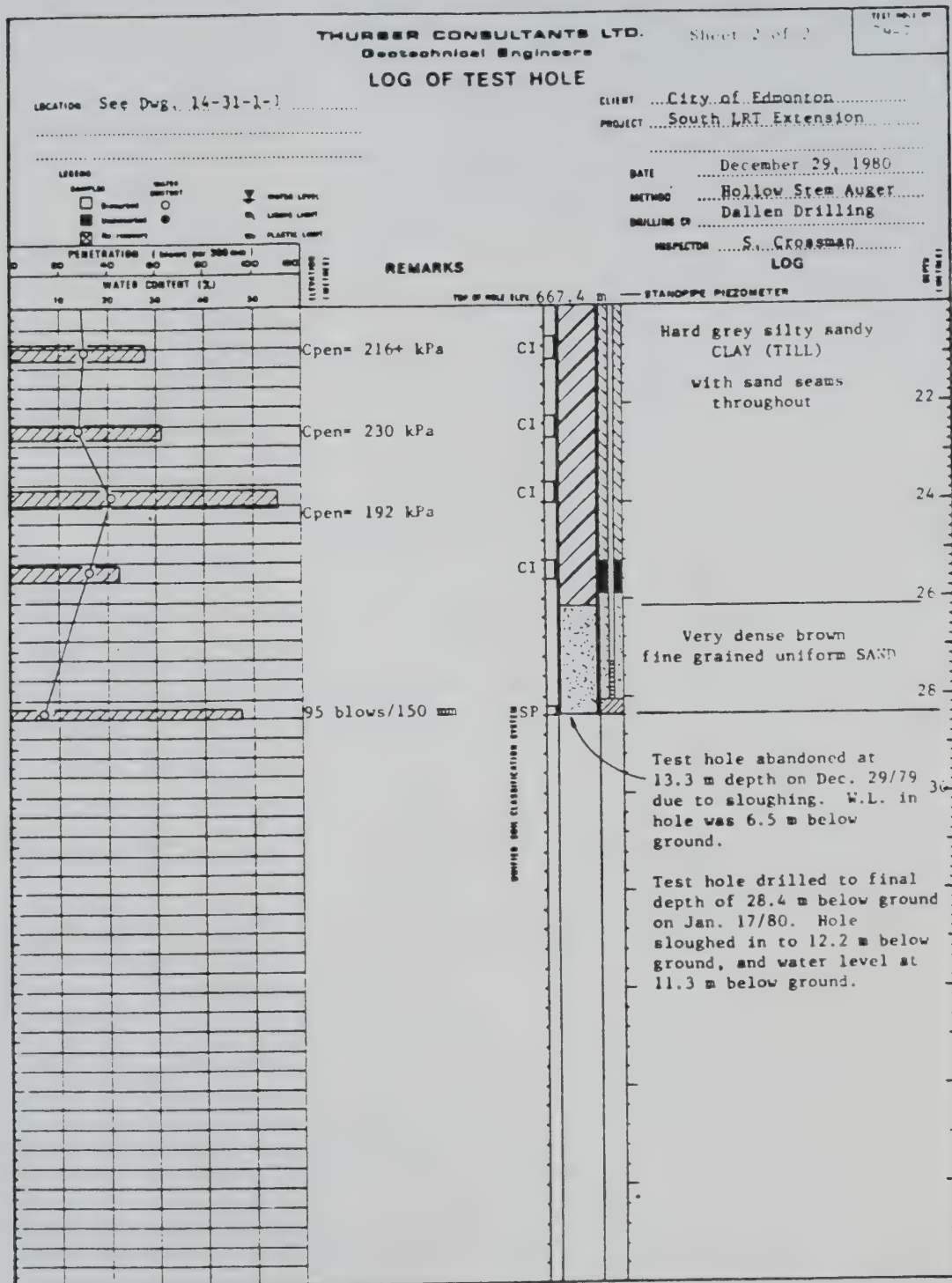
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Appendix C

Borehole test data





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